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SYSTEMS ANALYSIS OF IRRIGATION CONVEYANCE SYSTEMS

by

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DAVID H. MANZ

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Systems Analysis of Irrigation Conveyance Systems

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THE UNIVERSITY OF ALBERTA

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The undersigned certify that they have read, and recommended to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled **"Systems Analysis of Irrigation Conveyance Systems"** submitted by David H. Manz in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

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ABSTRACT

Procedures and techniques with which to describe all variations of irrigation conveyance systems and to evaluate their performance have been developed and demonstrated. The systems approach, defined in the most general sense, has been successfully applied. Terminology and classification systems have been developed with which to identify and describe the water demands irrigation conveyance systems are intended to satisfy, the physical components comprising irrigation conveyance systems, the subsystems of physical components comprising irrigation conveyance systems and the techniques used to operate irrigation conveyance systems. A performance criteria was developed with which to evaluate conveyance system performance. A computer model, named the Irrigation Conveyance System Simulation Model, was developed to enable conveyance system performance evaluation. The computer model was successfully demonstrated by considering a distributary canal typical of Southern Alberta, Canada.

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LIST OF SYMBOLS

Symbols	Description
A	cross-sectional area
b_{ck}	width of check
b_w	orifice width
B	width of channel of water surface
cp( )	structure control parameter
dfc	downstream flow condition
dsf( )	designer selected flow rate
dsh_u	designer selected upstream depth
d_w	orifice opening
$d_{w,max}$	maximum orifice opening
Δd_{ck}	width of stop-logs
e_c	conveyance efficiency
$e_{c,max}$	maximum conveyance efficiency
e_f	physical conveyance efficiency
e_o	operational efficiency
e_r	reservoir efficiency
fc( )	flow condition
F	Froude Number
g	acceleration due to gravity
h_{ck}	depth of water above check
$h_{ck,max}$	maximum height of check
i	rate of distributed lateral outflow
k_1	proportionality constant
k_2	constant

L	length of canal
n	Manning's "n"
$N(\ )$	pump motor speed
$obf(\ )$	observed flow rate through the structure
obh_u	observed depth of water upstream of depth-control structure
$osf(\ )$	operator selected flow rate through the structure
osh_u	operator selected depth of water upstream of the structure
$pcs(\ )$	adjustable physical characteristic of the structure affecting its hydraulic control
p	rate of distributed lateral inflow
Q	discharge
Q_1	flow rate immediately below check structure
Q_{ck}	flow past the check structure
Q_L	distributed lateral inflow or outflow
Q_N	flow rate immediately upstream of farm turnout
Q_W	flow through farm turnout
r_L	reservoir loss rate ratio
R	hydraulics radius
$Ra(\ )$	rating function for the structure
$sv(\ )$	state variable which governs the flow rate through structure
S_o	slope of channel bottom
S_f	slope of total energy line
t	actual time

t_o	actual time the structure is operated
u_f	unit physical conveyance efficiency
ufc	upstream flow condition
$v_{bc,c}$	volume of water diverted from reach for irrigation purposes
$v_{bc,r}$	volume of water delivered from reservoir for irrigation purposes
$v_{d,c}$	volume of drainage inflow to reach per unit length of reach
$v_{d,r}$	volume of drainage inflow to the reservoir
$v_{e,c}$	volume of evaporation outflow from reach per unit length of reach
$v_{e,r}$	volume of evaporation outflow from reservoir
$v_{o,c}$	volume of operational losses from reach
$v_{o,r}$	volume of operational losses from reservoir
$v_{p,c}$	volume of precipitation inflow to reach per unit length of reach
$v_{p,r}$	volume of precipitation inflow to the reservoir
$v_{s,c}$	volume of seepage outflow from reach per unit length of reach
$v_{s,r}$	volume of seeapge outflow from reservoir
$v_{t,c}$	volume of transpiration outflow from reach per unit length of reach
$v_{t,r}$	volume of transpiration outflow from reservoir
$v_{u,c}$	volume of controlled water inflow into reach
$v_{u,r}$	volume of inflow to the reservoir

$v_{st,r,fsL}$	volume of live storage at full supply level
$\Delta v_{st,c}$	change in volume of water stored per unit length of reach
$\Delta v_{st,r}$	change in volume of water stored in the reservoir
V	water velocity
V_1	water velocity downstream of check structure
V_N	water velocity immediately upstream of check
x	location along a reach as measured downstream from the entrance
y	water depth
y_1	water depth downstream of the check structure
y_N	water depth at node N
ΔZ_{ck}	minimum height of check structure
ΔZ_e	change in canal invert elevation
ΔZ_W	distance canal invert from bottom of orifice
θ	finite difference weighting parameter

CHAPTER 1

INTRODUCTION

Irrigation agriculture represents an important and frequently vital component of the local and regional economies of a substantial portion of the world. It is a very intensive form of agriculture which requires a large capital investment to initiate and which requires large expenditures to be operated, managed, and maintained. Irrigation agriculture is frequently the major and expanding consumer of limited water resources in a particular region.

The expansion of irrigation agriculture increases water demand. However, competition from other actual or potential water consumers, (municipal, industrial, environmental and recreational), for the available water is also increasing. Because the quantity of available water is often limited and expensive to develop for any potential water consumer, irrigation agriculture is being required to demonstrate that water is being used in the most efficient manner.

Irrigation agricultural activity consists of three principle components: water supply management, conveyance of water supply to the farmer consumer, and the application of the water to the land by the farmer consumer. Water supply management frequently forms part of the conveyance component. The water supply management and conveyance system components combined incur the majority of the capital and operational expenses associated with irrigation agricultural activity. These expenses are frequently distributed, via the tax-subsidy mechanisms, over the public as a whole and not, as might be anticipated, simply to the group of farmers who directly benefit from the availability of the water. Increasingly, society is requiring that it be demonstrated that the subsidies provided irrigation agriculture are being used in the most effective manner.

In summary, the irrigation communities throughout the world are, or soon will be, faced with the problems of:

1. demonstrating that they are using water in as efficient a manner as is reasonable to expect; and,
2. demonstrating that the particular irrigation agricultural activities which require substantial subsidization are as cost-effective as is reasonably possible.

The principle difficulty facing researchers who wish to develop the techniques with which the irrigation community may address the aforementioned requirements is the problem scope. Irrigation agriculture has been practised for thousands of years in a great variety of physical, economical, and cultural environments. As might be expected, existing irrigation conveyance systems display great physical and operational variability while satisfying many types of water demands. Substantial variability in the physical and operational characteristics of irrigation conveyance systems and the water demands they satisfy may be found within single autonomous irrigation projects, where a greater degree of uniformity and homogeneity of irrigation practises would normally be expected. The "variability" problem exacerbates the difficulties associated with the essential processes of evaluating and comparing the performance of different irrigation systems.

The complexity and variability of irrigation conveyance systems and the need to perform timely analysis suggests the use of modern computer facilities to simulate irrigation conveyance operations for purposes of performance evaluation. However, the difficulties inherent to the development of computer programs capable of simulating the operation of extensive conveyance systems demands that a systematic approach be taken to the computer program development in order to maximize program generality and thereby achieve the maximum benefit of the technique.

The formal "system analysis procedure" as described by Wagner (1975) was adopted. The procedure consists of the following steps:

1. Formulating the problem which yields a statement of the problem's elements. These include the controllable or decision variables, the uncontrollable variables, the restrictions or constraints on the variables and the objectives for defining a good or improved solution;
2. Building the model with which the variables are related;
3. Performing analyses using the model with which to address the problem under question; and,
4. Revise the model based on the findings of Step 3.

An extensive examination of available literature on irrigation conveyance systems was performed. This did not directly yield the problem formulation required on which to base the computer program development. A significant body of information was available, however, which described or suggested the water demands, physical components and systems, operational techniques and the performance evaluation criteria which could be used in the first step of a systems analyses of irrigation conveyance systems. Simultaneous with the examination of literature on the physical and operational characteristics of irrigation conveyance systems, techniques were being sought which could be used to simulate the various hydraulic and hydrologic processes associated with the conveyance systems components identified. It was determined that sufficient techniques existed for the second step in the systems analyses of irrigation conveyance systems, and that simulation of irrigation conveyance systems operation using computer modelling techniques was practicable, Manz (1982). It also became apparent that the development of a completely general computer model, capable of simulating all types and varieties of irrigation conveyance systems was beyond the scope of the research endeavor being undertaken. It was decided, therefore, to develop a limited model not only capable of demonstrating the utility of the descriptive techniques, but which also would provide the basic structure for general versions to be developed in the future.

All four of the steps to the system analysis of irrigation conveyance systems as suggested by Wagner (1975) were performed. Methods and terminology have been developed by which virtually every variation of irrigation conveyance system water demand, physical characteristic and operational procedure can be identified and described. Criteria were developed to evaluate irrigation conveyance system performance. A computer model was developed which was capable of simulating the physical and operational behavior of irrigation conveyance systems physical and operational behaviour and of providing the information necessary to apply the performance criteria. The computer model was demonstrated on a distributary canal typical of those found in Southern Alberta, Canada.

The research reported is divided into four segments. The first segment consists of the development and discussion of terminology and classification methods with which to identify and describe the farm water demands conveyance systems are intended to satisfy, individual irrigation conveyance system components, systems and subsystems of irrigation conveyance system components and the methods used to operate the various irrigation conveyance systems and subsystems. The second segment consists of the development and description of criteria with which irrigation conveyance systems may be evaluated. The third segment consists of the description and demonstration of the computer model developed to simulate irrigation conveyance system operations. The fourth segment consists of the description of the theoretical hydraulics and hydrology employed by the model. For purposes of readability only, segments one, two, and three are contained in the body of the text and the fourth segment in the appendices.

CHAPTER 2

OBJECTIVES

The specific objectives of the research undertaken are as follows:

1. to identify and describe the water demands irrigation conveyance systems are intended to satisfy;
2. to identify and describe the physical components which comprise irrigation conveyance systems;
3. to identify and describe subsystems of physical components to enable the description of irrigation conveyance system configuration;
4. to describe irrigation conveyance system operations in relation to the water demand they are intended to satisfy, their physical characteristics and the environment they are operated in;
5. to develop performance criteria with which the efficiency of irrigation conveyance systems, subsystems, and components may be evaluated and compared; and,
6. to develop and demonstrate a computer model which would enable the evaluation of irrigation conveyance system performance while considering most of the pertinent physical, environmental, and operational characteristics of the system.

CHAPTER 3

FARM WATER DEMAND

3.1 Introduction

Implicit to the development of irrigation conveyance system design and operational procedure is the definition of the farm water demand the conveyance system is to satisfy. The exact nature of the farm water demand is determined by the type of agricultural practises employed and the availability of water for irrigation purposes. Though farm water requirements may be estimated and scheduled, Jensen (1973), Doorenbos and Pruitt (1977), Burman, et. al. (1980) and Stegman, et. al. (1980), the actual water demands the farmers will have will also be determined by their agricultural skills and a variety of economic, social, political and environmental factors, Pohjakas (1978) and (1981).

Several attempts have been made to classify the numerous forms of farm water demand. The oldest reference available to the writer is Harding (1917) who bases his work on studies and reports of still earlier vintage. More recent attempts to classify farm water demands are reported by Repogle, et al. (1980), Repogle and Merriam (1980) and Sagardoy, et al. (1982). Though none of these studies can be considered comprehensive, they do suggest the means by which a comprehensive terminology for describing farm water demands can be achieved.

Results from two of the studies, Repogle and Merriam (1980) and Sagardoy, et al. (1982), are presented. (The classification system presented by Repogle and Merriam (1980) is essentially an expanded version of that presented by Repogle, et al. (1980).)

3.2 Repogle and Merriam (1980)

Repogle and Merriam (1980) named the various types of farm water demands "irrigation schedules" which they define as follows: "Irrigation schedules are the expressions in terms of frequency, rate and duration of how water is diverted or delivered to a farm unit". They further indicate that the schedules may be "supplier-controlled or user-controlled" and "rigid or flexible". It should be noted that "control" implies decision making authority and is not synonymous with physical operation. Table 3.1 summarizes their work. (It should be noted that a schedule whose name includes the word demand indicates that the farmers physically operate their own water supply.)

The modifiers in the frequency, rate and duration columns are defined by Repogle and Merriam as follows:

1. Unlimited - Unlimited and controlled by user.
2. Limited - Maximum flow rate limited by physical size of system or turnout capacity but causing only moderate to negligible problems in farm operations. The applied rate is controlled by the user and may be varied as desired.
3. Arranged - Day or days of water availability are arranged between the water agency and the user.
4. Constant - The condition or rate or duration remains constant as arranged during the specific irrigation run.
5. Fixed - The condition is predetermined by the agency.

Each type of schedule is briefly described.

Table 3.1. Schedule Constraints

Schedule Name	Frequency	Rate	Duration
1. Demand	Unlimited	Unlimited	Unlimited
2. Limited-rate, demand	Unlimited	Limited	Unlimited
3. Arranged	Arranged	Unlimited	Unlimited
4. Limit-rate, arranged	Arranged	Limited	Unlimited
5. Restricted arranged	Arranged	Constant	Constant
6. Fixed duration, restricted arranged	Arranged	Constant	Fixed by Policy
7. Varied amount, constant frequency, (modified amount rotation)	Fixed	Varied as Fixed	Fixed
8. Constant amount, varied frequency, (modified frequency rotation)	Varied as Fixed	Fixed	Fixed
9. Constant amount, constant frequency, (rotation)	Fixed	Fixed	Fixed

3.2.1 Demand

Water is withdrawn from the conveyance system at the frequency, rate and duration of the farmer's choice. This may be possible where the water supply is a large reservoir or large canal where the farmer's requirements are insignificant compared to the large volumes or flows of water.

3.2.2 Limited-rate Demand

Limited-rate demand delivery methods differ only from the demand-type in the sense that the flow rates through each farm turnout are limited. This is usually a negligible constraint since most farm turnouts are constructed such that their capacities far exceed foreseeable farm requirements for water.

3.2.3 Arranged

The arranged delivery method requires that the farmer simply request that water be made available to him at the rate and duration of his choice. The farmer normally communicates the fact that he needs water to conveyance system operating personnel a few days in advance of when he requires it. The conveyance system operating personnel arrange for sufficient water to be available as long as he requires it.

3.2.4 Limited-rate, Arranged

The limited-rate, arranged method differs from the arranged method only in so far as an upper limit is set on the flow through the farm turnouts. Flow rate and duration are farmer controlled.

3.2.5 Restricted Arranged

The restricted arranged method is similar to the arranged method except that the rate and duration of water deliveries, once set, are held constant throughout a specific water delivery. Duration may vary with each water delivery during the irrigation season.

3.2.6 Fixed Duration, Restricted Arranged

This delivery method differs from the restricted arranged method in that the duration of deliveries are fixed by arrangement prior to the irrigation season.

3.2.7 Varied Amount, Constant Frequency

This is a form of rotation water delivery. The farmer will receive water at variable flow rates agreed upon at the beginning of an irrigation season at fixed intervals and durations. The canal supplying the farmers always flows full with each farmer taking his turn receiving water.

3.2.8 Constant Amount, Varied Frequency

This is a form of a rotation schedule. The amount of water applied per rotation is constant but the frequency of delivery varies throughout the irrigation season.

3.2.9 Constant Amount, Constant Frequency

This delivery method is also known as the rotation schedule. Continuous flow canals would be a limiting case. Each farmer takes his turn withdrawing water from the canal at constant frequency and rate.

3.3 Sagardoy, Bottrall and Uittenbogaard (1982)

Sagardoy, et al. (1982), have identified five main types of farm water demand which they call water distribution methods. They are defined as follows:

3.3.1. On demand

Water is available to the farmer any time that the intake or hydrant is opened. Therefore, the amounts to be used are not limited but water consumption is usually metered and paid for by volume.

3.3.2 Semi-demand

Water is made available to the farmer within a few days (generally 2 - 7 days) of his request. The amount is often limited to a certain volume per unit area.

3.3.3 Canal rotation and free demands

Secondary canals receive water by turns, for example every 7 days; and, once the canal has water, farmers can take the amount they need at the time they wish.

3.3.4 Rotational system

Secondary canals receive water by turns and the individual farmers within a given canal area receive the water at a pre-set time and generally in a limited quantity.

3.3.5 Continuous flow

Throughout the irrigation season, the farmer receives a small but continuous flow that compensates for daily crop evapotranspiration.

3.4 Recommended Terminology for Describing Farm Water Demand

The work reported by Repogle and Merriam (1980) is particularly significant in that the basic characteristics of farm water demand are identified; specifically, frequency, rate and duration. These characteristics may be defined as follows:

1. Frequency of demand refers to the time a water delivery may start.
2. Rate of water demand describes the flow rate associated with a particular delivery.
3. Duration describes the length of time a particular delivery will last.

Merriam and Repogle (1980) also indicate that "irrigation schedules", (i.e. farm water demand), may be rigid or flexible and that they may be supplier or user controlled. They introduce a number of modifiers to describe these quantities as indicated in Table 3.1.

It is proposed that the following modifiers be used to express the relative "rigidity" or "flexibility" of the three fundamental characteristics of farm water demand:

1. Unlimited flexibility which implies that the farm water demand characteristic, (i.e. frequency, rate, or duration) is unconstrained within the broadest range the user could reasonably require for irrigation purposes.
2. Limited flexibility which implies that the farm water demand characteristic is unconstrained within a limited range.
3. Inflexible which implies that the farm water demand characteristic is constant throughout irrigation season. (Note that an irrigation season may include several irrigation runs or periods where water is applied to the land.)
4. Unlimited varied as fixed flexibility which implies that the farm water demand characteristic may be different for each run but is held constant during each run. The selection of farm water demand characteristic is unconstrained within the broadest range the user could reasonably require for irrigation purposes.
5. Limited varied as fixed flexibility which implies that the farm water demand characteristic may be different for each irrigation run but is held constant during each run. The

selection of farm water demand characteristic is unconstrained within a limited range.

It is further proposed that control over farm water demand flexibility be classified into one of three types; user, collaborative, and agency. (Agency refers to a water supply management organization which may be responsible for delivering water to the user, the farmer.) Each type of control is defined as follows:

1. User control implies that the selection of water demand characteristic is determined solely by the farmer. No agency input need be considered.
2. Collaborative control implies that the selection of water demand characteristic is the joint decision of the farmer and the agency; Repogle and Merriam (1980) use the term "arranged" to indicate collaborative control.
   (Types of farm water demand which are farmer or collaboratively controlled are said to use "demand control", Moore (1981).)
3. Agency control implies that the selection of water demand characteristic is determined solely by the agency. No farmer, (user), input need be considered.
   (Types of farm water demand which are agency controlled are also called "supplier-controlled", Moore (1981) and Wade (1981).)

3.5 Application of Recommended Terminology for Describing Farm Water Demand

It is not reasonable to attempt to list and discuss each of the several hundred possible variations of farm water demand suggested by the proposed terminology. In order to demonstrate the utility of the proposed terminology, however, it will be used to describe each type of farm water demand identified by Merriam and Repogle (1980) shown in Table 3.1 and identified by Sagardoy et al. (1982). The results of this application are shown in Tables 3.2 and 3.3 respectively.

It is evident from examination of the analyses of the farm water demand terminology reported by Merriam and Repogle (1980) and by Sagardoy et al. (1982), that the proposed terminology adequately describes the character of farm water demand from the perspective of conveyance system operations. Use of the terminology actually suggests variations which otherwise may not have been recognized. It is recommended, however, that the names given to individual types of farm water demands be retained because of the additional quantitative information the descriptions usually contain.

3.6 Discussion

The importance of concisely defining the nature of the farm water demand irrigation conveyance systems are intended to satisfy cannot be overemphasized. As stated in the introduction to this Chapter, the determination and/or selection of a particular form of farm water demand will determine, to a significant extent, available design and

TABLE 3.2
Analysis of Farm Water Demand Terminology Reported by
Repogle and Merriam (1980)

Merriam and Repogle	Description Using Proposed Terminology		
Schedule Name	Frequency	Rate	Duration
1. Demand	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility
2. Limited-rate demand	Farmer controlled Unlimited flexibility	Farmer controlled Limited flexibility	Farmer controlled Unlimited flexibility
3. Arranged	Collaboratively controlled Unlimited flexibility	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility
4. Limited-rate arranged	Collaboratively controlled Unlimited flexibility	Farmer controlled Limited flexibility	Farmer controlled Unlimited flexibility
5. Restricted arranged	Collaboratively controlled Unlimited flexibility	Collaborative Control Unlimited or Limited varied as fixed flexibility	Collaborative control Unlimited or Limited varied as fixed flexibility
6. Fixed-duration restricted arranged	Collaboratively controlled Unlimited flexibility	Collaborative Control Unlimited or Limited varied as fixed flexibility	Agency controlled Inflexible
7. Varied amount, constant frequency	Agency controlled Inflexible	Farmer/collaborative/agency control, Limited varied as fixed flexibility	Agency controlled Inflexible
8. Constant amount, varied frequency	Farmer/ collaborative/agency control, Limited varied as fixed flexibility	Agency controlled Inflexible	Agency controlled Inflexible
9. Constant amount, constant frequency	Agency controlled Inflexible	Agency controlled Inflexible	Agency controlled Inflexible

TABLE 3.3
Analysis of Farm Water Demand Terminology
Reported by Sagardoy et al. (1982)

Sagardoy et al. (1982)	Description Using Proposed Terminology		
Method of Water Distribution	Frequency	Rate	Duration
1. On-demand	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility
2. Semi-demand	Collaborative control Unlimited flexibility	Collaborative control[1] Limited varied as fixed flexibility	Collaborative control[1] Limited varied as fixed
3. Canal rotation and free demand	Agency controlled Inflexible	Farmer controlled Unlimited flexibility	Farmer controlled Unlimited flexibility
4. Rotational system	Agency controlled[2] Inflexible	Agency controlled[2] Inflexible	Agency controlled[2] Inflexible
5. Continuous flow	Agency controlled Inflexible	Agency controlled Inflexible	Agency controlled Inflexible

[1] volume limited

[2] collaborative control and more flexibility may be possible within the general definition provided for rotational method of water distribution.

operational options. For any particular conveyance system, the specific nature of the farm water demand it intends to satisfy and the water loss/gain characteristics of the system, together will determine the characteristics of all required water diversion operations. The operation of irrigation conveyance systems is discussed in detail in Chapter 6.

CHAPTER 4

CLASSIFICATION OF IRRIGATION CONVEYANCE SYSTEM COMPONENTS

4.1 Introduction

The proposed system for classification of irrigation conveyance system components is intended to provide a method with which to identify, describe and compare each of the numerous and diverse structural components found within irrigation conveyance systems. The proposed classification system considers both the hydraulic and operational characteristics of the components; however, the role each component serves in the management and/or control of water flows either into, within or out of the conveyance system is emphasized.

The primary level of classification considered is the water management function the conveyance system component is intended to serve. Four water management functions are identified: diversion, transfer, depth-control and storage. Subsequent levels of classification are based on component hydraulic and operational characteristics and are specific to each of the four water management functions identified.

Diversion and depth-control structures are classified according to the type of hydraulic control governing flow through the structure, method of structure operation, type of control incorporated in the method used to operate the structure and the level to which the structure's operation is co-ordinated with the operation of other components in the conveyance system.

Transfer structures are classified into the following categories; canals, pipelines, canal-associated specialty structures and pipeline-associated specialty structures. Canals are classified according to method of construction. Pipelines are classified according to whether they are open or closed systems. Canal- and pipeline-associated structures are both classified according to the specific specialty function they are intended to serve.

Storage facilities are classified according to the principal use they are intended to serve. Three categories are identified.

Conveyance system components which combine more than one water management function into a single or compound structure are also considered.

General equations were developed which describe the hydraulics and operation of each of the diversion, depth-control and transfer components identified.

Information pertaining to the design and operational characteristics of the majority of the components used in irrigation conveyance systems may be found in the following standard references: USBR (1967), (1974), and (1977), Smith (1978), Kraatz and Mahajan (1975), Chow (1959), Henderson (1966), Houk (1956), Etcheverry (1915(b) and 1915(c)), ICID (1979) and Framji (1972). This body of literature also contains information on a variety of other structures not considered in the classification system presented because these structures, such as bridges, cross-drainage, safety, and so on, do not affect conveyance system operations.

4.2 Diversion Structures

4.2.1 Water Management Function

Diversion structures are used to control the inflow or outflow of water to or from any part or all of the irrigation conveyance system. Specifically, diversion structures are used to:

1. control the flow of water from the primary water supply into the conveyance system;
2. control the flow of water from one portion of the conveyance system into any other portion of the conveyance system;
3. control the flow of water from the conveyance system to where the water is applied to the land;
4. control the flow of water out of the conveyance system to where it is no longer available to any other portion of the conveyance system or available for application to the land; and,
5. control the flow of water from sources other than the primary water supply into the conveyance system (eg. storm runoff).

4.2.2 Classification System

Diversion structures may be classified according to the type of hydraulic control which governs the flow of water through the structure, the method used to operate the structure, the type of control incorporated in the method used to operate the structure and the degree with which the operation of the individual diversion structure is co-ordinated with the operation of other structures, flow conditions or water demands within the conveyance system.

4.2.2.1 Hydraulic Control

Four types of hydraulic control are identified; positive control, upstream control, downstream control and upstream/downstream control.

4.2.2.1.1 Positive

Positive hydraulic control implies that flow through the structure is virtually independent of flow conditions in the conveyance system. Pump-motor systems are frequently used in situations where they may be classified as positive hydraulically controlled.

4.2.2.1.2 Upstream

Upstream hydraulic control implies that the flow of water through the diversion structure is solely dependent on flow conditions upstream of the diversion structure. Examples of diversion structures with upstream hydraulic control include the many types of weirs and free-outlet type orifices.

4.2.2.1.3 Downstream

Downstream hydraulic control implies that the flow of water through the diversion structure is solely dependent on flow conditions downstream of the structure. An example would be an orifice operating with a submerged outlet where the upstream depth is essentially constant and the downstream depth variable.

4.2.2.1.4 Upstream/Downstream

Upstream/downstream hydraulic control implies that the flow of water through the diversion structure is dependent on flow conditions upstream and downstream of the structure. An example is an orifice operating with a submerged outlet and the upstream and downstream depths are both variable.

4.2.2.2 Method of Operation

Diversion structures may be classified according to method of operation; that is, according to the operability of the structure as determined by the extent of operator participation permitted in the selection of flow through the structure; and, by the operator's ability to physically adjust and thereby modify the hydraulics governing flow through the structure. Four different methods of operation are identified: manual/remote, automatic, fixed and fixed-automatic.

4.2.2.2.1 Manual/Remote

Manual/remote operation of a diversion structure implies that there is direct operator participation in selecting the flow through the structure and in the physical adjustment of the structure. Manual and remote operation differ only to the extent that operator presence at the structure is not required for remote operation. Remote interrogation and/or reporting of flow conditions in the vicinity of the diversion structure or the physical status of diversion structure should not be confused with remote operation.

4.2.2.2.2 Automatic

Automatic operation of a diversion structure implies that there is direct operator participation, on site or remote, in selecting the flow through the structure but the physical adjustment of the structure to achieve the selected flow is performed by a controller-actuator system incorporated into the structure itself. The controller receives information characterizing the state of the flow condition structure system. It compares this state to some target value previously selected by the operator. Depending on the difference between the selected target value and the controller observed or calculated states and the specific nature of the control the system is using, the controller, using the actuator, will physically adjust the diversion structure and the flow of water through it.

4.2.2.2.3 Fixed

Fixed diversion structures are not operable. No operator participation is possible either in selecting the flow through the structure or in the physical adjustment of the structure. Not only is there no provision for adjustment to take place; but also, all structural elements are fixed in position. The hydraulic characteristics of the diversion structure are completely established at the time of construction. Flow through the structure is totally dependent on the flow conditions occurring during the time the structure is commissioned.

4.2.2.2.4 Fixed-Automatic

Fixed-automatic diversion structures are also not operable. No operator participation is possible in selecting the flow through the structure or in the physical adjustment of the structure. The structural elements which govern flow through the structure are not fixed however but the structure is automatically controlled. The selection of target flows through the structure and mode of control have been made and fixed at the time the structure was commissioned.

4.2.2.3 Control of Operation

Diversion structures may be classified according to the type of control incorporated into the particular method used to operate the structure. Two types are identified: open-loop control and closed-loop control.

4.2.2.3.1 Open-Loop

Open-loop control implies that the flow of water through the diversion structure is determined by the relative positioning of appropriate structural elements within the diversion structure itself. Diversion structures which use open-loop control may use any amount of flow condition information or information on the position of particular structural elements in a rating function to determine the water flow through the structure.

4.2.2.3.2 Closed-Loop

Closed-loop control implies that flow through the diversion structure is determined using observations of flow rates through the structure made at the time the structure is operated. Adjustment of the diversion structure continues until the observed flow rate corresponds to the desired or selected flow rate.

4.2.2.4 Co-ordination of Operation

Individual diversion structures may be further classified according to how their operation is co-ordinated with the operation of other structures' flow conditions or water demands within the conveyance system. Three degrees of co-ordination are identified: system, local and independent.

4.2.2.4.1 System

Co-ordination of operation at the system level implies that the operation of the diversion structure is co-ordinated with the operation of other parts of the conveyance system. The structure is operated in response to water demands but not without consideration of system structure and flow conditions and other water demands which may be placed on the system. Most diversion structures are system co-ordinated.

4.2.2.4.2 Local

Co-ordination of operation at the local level implies that the operation of the diversion structure does not consider operations, flow conditions or water demands elsewhere in the conveyance system. The operator satisfies local water demands as required. An example of a locally co-ordinated diversion structure is a low-capacity gated orifice operating off a large internal reservoir.

4.2.2.4.3 Independent

Independently co-ordinated diversion structures operate similar to locally co-ordinated structures but without operator involvement. An example of an independently co-ordinated diversion structure is a storm drain into a canal which operates only when the adjacent land area receives rainfall.

4.2.2.5 Complex Diversion Structures

Complex diversion structures incorporate several independently functioning outlets. Each outlet is equivalent to a single simple diversion structure and may be classified and described accordingly. Outlets corresponding to the same classification or to several different classifications of diversion structure may be incorporated into a single complex structure. All outlets discharge into the same canal or pipeline.

4.2.3 General Equations Describing the Operation of Diversion Structures

General equations are presented which describe the operation of diversion structures identified in the preceding sections. The equations are grouped according to the method of control used to operate the structure. Co-ordination of diversion structure operation is not considered.

4.2.3.1 Open-Loop Controlled Diversion Structures

General equations describing the operation of open-loop controlled diversion structures are presented as follows.

Type 1 - Positive hydraulic control
- Manual/remote method of operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), sv(t_0), fc(t_0), cp(t_0)), sv(t)) \qquad (4.1)$$

where: $osf(t_0)$ is the operator selected flow rate through the structure at time of operation, t_0.

$sv(t_0)$ is the state variable which governs the flow rate through structure considered by the operator at time of operation, t_0. There may be more than one $sv(t_0)$.

$sv(t)$ is the state variable at any time, t. There may be more than one continually varying state variable. Note that the state variables considered by the operator will not necessarily include all of the state variables determining the flow through the structure.

$fc(t_0)$ is the flow condition which is considered at the time the structure is operated. More than one $fc(t_0)$ may be considered.

$cp(t_0)$ is the structure control parameter at the time the structure is operated. More than one $cp(t_0)$ may be included.

pcs( ) is the adjustable physical characteristic of the structure affecting its hydraulic control. Normally, there is only one pcs-function though several are possible.

Ra( ) is the rating function for the structure. The rating function permits the calculation of actual water flow rate through the structure, Q(t).

t_0 is the actual time the structure is operated.

t is the actual time; not necessarily since the time the structure was operated.

Q(t) is the actual water flow rate.

The osf and pcs functions are interrelated as indicated by the following equations:

$$osf(t) = f_1(pcs(t), sv(t)) \quad (4.2(a))$$

and $$pcs(t) = f_2(osf(t), sv(t)) \quad (4.2(b))$$

Under ideal circumstances, the f_1-function is identical to the Ra-function in Equation 4.1. In reality, however, considerable differences between the two functions may exist.

An example of a Type 1 diversion structure is a manually operated pump-motor assembly, where the pump discharge is a function of motor speed. The operator controls the pump discharge by controlling motor speed, N. In equation form:

$$Q(t) = Ra(N(t, osf)) \tag{4.3}$$

In this example

$$N(t, osf) \equiv pcs(\ ) \tag{4.4}$$

In this example, variations in other factors; that is, state variables, which could affect pump performance, are assumed negligible.

Examples of pump-motor assembly associated state variables include those factors which affect the characteristics of pump-performance curve itself or motor speed. State variables, (e.g. electric power supply), do not usually vary significantly with time.

Pump-motor assembly associated control parameters include those factors which determine the way the operator will physically adjust or operate the system. For example, start-up procedures would be defined using control parameters.

An example of a flow condition which would normally be considered at the time a pump-motor system is operated is the available water supply, usually indicated by upstream water depth. Note that in the example of the pump-motor assembly just discussed, it is assumed that flow conditions do not significantly affect pump discharge characteristics. If, for, example, suction head influenced pump discharge, the pump-motor assembly would no longer be considered to operate under positive hydraulic control. Instead, it would be classified as operating under upstream hydraulic control.

Type 2 - Positive hydraulic control
- Automatic method of operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), sv(t_0, t), fc(t_0, t), cp(t_0, t)), sv(t)) \quad (4.5)$$

where: $osf(t_0, t)$ is the operator selected flow rate through the structure supplied to the controller at the time of operation, t_0, and possibly modified by it anytime thereafter according to algorithms fully understood and used by the operator.

$sv(t_0, t)$ is the state variable which may be considered by the operator at the time the flow rate through the structure is selected and the state variable considered anytime thereafter by the controller. More than one $sv(t_0, t)$ may be considered. However, the state variables considered by the operator and the controller will not necessarily include all of the state variables determining flow through the structure.

$fc(t_0, t)$ is the flow condition which may be considered at the time the flow rate through the structure is selected and the flow condition considered anytime thereafter by the controller. More than one $fc(t_0, t)$ may be considered.

$cp(t_0, t)$ is the structure control parameter considered at the time the flow rate through the structure is selected and anytime thereafter by the controller. More than one control parameter may be considered.

Type 2 diversion structures have limited application within irrigation conveyance systems. Usually, if positive hydraulically controlled diversion structures are used, they employ closed-loop operation control. That is, they are Type 18 diversion structures.

An example of a Type 2 diversion structure would be a pump-motor assembly which automatically starts and stops operation at specific times each day. The pump discharge and the times and manner with which the pump starts and stops are selected by the operator. Flow through the pump-motor assembly just described may be represented by the equation:

$$Q(t) = Ra(N(osf(t_0), cp(t_0, t)) \quad (4.6)$$

and $$N(\ ) \equiv pcs(\ ) \quad (4.7)$$

Note that the state variables are assumed unoperable and constant. Also, note that $cp(t_0, t)$ is used to set the time and manner the pump starts and stops.

Type 3 - Positive hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs, sv(t)) \quad (4.8)$$

The physical characteristics of the structure which determine the nature of the flow through the structure are fixed. Flow variations through the structure are caused by variation in the state variables.

An example of a Type 3 diversion structure is a pump-motor assembly which lifts water from a reservoir to a canal and where the total lift remains constant with time. The motor driving the pump is simply started and left to run. Variations in pump discharge would occur due to variations in power supply or pump performance characteristics.

Type 4 - Positive hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the equation:

$$Q(t) = Ra(pcs(dsf(t), sv(t), fc(t), cp(t)), sv(t)) \qquad (4.9)$$

where: dsf(t) is the designer selected flow rate which is supplied to the controller at the time the structure was commissioned. The dsf may vary with time but cannot be modified by the operator.

sv(t) is the state variable governing flow through the structure and considered by the controller operating the structure. More than one sv may affect the rate of flow through the structure or be considered by the controller operating the structure. The state variables affecting the flow through the structure may not all be considered by the controller.

cp(t) is the structure control parameter which may be fixed at the time of construction or which may vary with time. More than one control parameter may be considered.

fc(t) is the flow condition which is considered by the controller prior to structure operation. More than one flow condition may be considered.

Type 4 diversion structures operate in the same manner as Type 2 structures with the exception that osf(t_0, t), sv(t_0, t), fc(t_0, t) and cp(t_0, t) are replaced with dsf(t), sv(t), fc(t) and cp(t) respectively. Type 4 diversion structures have limited applicability in irrigation conveyance systems.

An example of a Type 4 diversion structure is the automatically operated pump-motor assembly used as an example of a Type 2 diversion structure with the modification that the osf(t_0, t) and cp(t_0, t) are replaced with a dsf(t) and cp(t).

Type 5 - Upstream hydraulic control
- Manual/remote operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), fc(t_0), cp(t_0), ufc(t)) \qquad (4.10)$$

where: ufc(t) is the upstream flow condition which governs the flow rate through the structure. More than one ufc may be operative.

An example of a Type 5 diversion structure is a gated orifice which has a free outlet. The $fc(t_0)$ and $ufc(t)$ are equal to the water depth upstream of the structure at times t_0 and t respectively.

Type 6 - Upstream hydraulic control
- Automatic operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t)) \tag{4.11}$$

An example of a Type 6 diversion structure may be constructed by automating the gated orifice with free outlet provided as an example of a Type 5 diversion structure. This is achieved by providing the controller with the appropriate pcs-function and the upstream depth on a continuous basis. The controller will operate the actuator in order to adjust the gate opening to maintain the desired $osf(t_0, t)$.

Type 7 - Upstream hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs, ufc(t)) \tag{4.12}$$

Examples of Type 7 diversion structures include fixed height weir-type spillways and syphon spillways.

Type 8 - Upstream hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsf(t), fc(t), cp(t)), ufc(t)) \qquad (4.13)$$

The example provided for a Type 6 diversion structure may be converted to a Type 8 structure by replacing the $osf(t_0, t)$ with a $dsf(t)$.

Type 9 - Downstream hydraulic control
- Manual/remote operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), fc(t_0), cp(t_0)), dfc(t)) \qquad (4.14)$$

where: $dfc(t)$ is the downstream flow condition governing the flow rate through the structure. More than one dfc may be operative.

An example of a Type 9 diversion structure is a gated orifice with a submerged outlet which controls the discharge from a reservoir whose depth remains relatively constant. The $dfc(t)$ is the depth of water downstream of the diversion structure.

Type 10 - Downstream hydraulic control
- Automatic operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), fc(t_0, t), cp(t_0, t), dfc(t)) \qquad (4.15)$$

An example of a Type 10 diversion structure may be constructed by automating the gated orifice with submerged outlet provided as an example of a Type 9 structure. This is achieved by providing the controller with the appropriate pcs-function, the observed depth of reservoir; and, on a continuous basis, the water depth downstream of the structure. The controller will operate the actuator in order to adjust the gate opening to maintain the desired osf.

Type 11 - Downstream hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs, dfc(t)) \qquad (4.16)$$

An example of a Type 11 diversion structure is a siphon with a submerged inlet and outlet. The siphon action is never broken as the inlet of the siphon is always submerged by a fixed depth of water, (e.g. by a large continuous flow canal); and, the outlet of the siphon is submerged in a pond. Once the siphon is primed, the siphoning action will commence whenever the pond water level drops below that of the canal.

Type 12 - Downstream hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsf(t), fc(t), cp(t)), dfc(t)) \tag{4.17}$$

The example provided for a Type 10 diversion structure may be converted to a Type 12 by replacing the $osf(t_0, t)$ with a $dsf(t)$.

Type 13 - Upstream/downstream hydraulic control
- Manual/remote operation
- Open-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), fc(t_0), cp(t_0)), ufc(t), dfc(t)) \tag{4.18}$$

An example of a Type 13 diversion structure is a gated orifice with a submerged outlet delivering water from a canal, which has a varying water depth, into another canal, which also has a varying depth.

Type 14 - Upstream/downstream hydraulic control
- Automatic operation
- Open-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t), dfc(t)) \quad \ldots \tag{4.19}$$

An example of a Type 14 diversion structure may be constructed by automating the gated orifice with submerged outlet provided as an example of a Type 13 structure. This could be achieved by providing the controller with the appropriate pcs-function and the upstream and downstream water depths on a continuous basis. The controller would operate the actuator in order to adjust the gate opening to maintain the desired osf.

Type 15 - Upstream/downstream hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs, ufc(t), dfc(t)) \tag{4.20}$$

An example of a Type 15 diversion structure is an ungated orifice with a submerged outlet delivering water from one canal of variable depth to another canal also of variable depth.

Type 16 - Upstream/downstream hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

$$Q(t) = Ra(pcs(dsf(t), fc(t), cp(t)), ufc(t), dfc(t)) \tag{4.21}$$

The example provided by a Type 14 diversion structure may be converted into a Type 16 structure by replacing the $osf(t_0, t)$ with a dsf(t).

4.2.3.2 Closed-Loop Operation Controlled Diversion Structures

General equations describing the flow through closed-loop controlled diversion structures are presented as follows:

Type 17 - Positive hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), obf(t_0), fc(t_0), cp(t_0), sv(t_0)), sv(t)) \quad \ldots (4.22)$$

where: $obf(t_0)$ is the observed flow rate through the structure at time of operation.

An example of a Type 17 diversion structure is a manually operated pump-motor assembly which employs a flow measurement device on the downstream line. The operator will adjust the motor speed until the observed flow matches the operator's selected flow. An important state variable which may control the flow rate through the pump is downstream line pressure.

Type 18 - Positive hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through this type of diversion structure may be described using the equation:

$$Q(t) = Ra(pcs(osf(t_0, t), obf(t_0, t), fc(t_0, t), cp(t_0), sv(t_0)), sv(t)) \quad \ldots (4.23)$$

An example of a Type 18 diversion structure is an automated Type 17. This may be achieved by providing the controller with the appropriate pcs-function and the observed downstream flow rate. The controller will compare the osf and obf and will operate the actuator to adjust motor speed until the osf and obf match.

Type 19 - Positive hydraulic control
- Fixed operation
- Closed-loop control of operation

No examples of this type of diversion structure exist.

Type 20 - Positive hydraulic control
- Fixed-automatic operation
- Closed-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsf(t), obf(t), fc(t), cp(t), sv(t)), sv(t)) \quad (4.24)$$

The example provided for Type 18 diversion structures may be converted to a Type 20 by replacing the osf-function with a dsf.

Type 21 - Upstream hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(psc(osf(t_0), obf(t_0), fc(t), cp(t_0)), ufc(t_0)) \quad (4.25)$$

An example of a Type 21 diversion structure is a gated orifice with a free outlet where there is provision to make observations of flow rate through the structure. The operator will adjust the gate until the osf matches the obf.

Type 22 - Upstream hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), obf(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t)) \quad \ldots (4.26)$$

An example of a Type 22 diversion structure is an automated Type 21 structure. This may be achieved by providing the controller with the appropriate psc-function and observed downstream flow information. The controller will compare the osf and the obf and will operate the actuator to adjust the gate opening until the osf and the obf match.

Type 23 - Upstream hydraulic control
- Fixed operation
- Closed-loop control of operation

No examples of this type of diversion structure exist.

Type 24 - Upstream hydraulic control
- Fixed-automatic operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described by the equation:

$$Q(t) = Ra(pcs(dsf(t), obf(t), fc(t), cp(t)), ufc(t)) \quad (4.27)$$

The example provided for a Type 22 diversion structure may be converted to a Type 24 structure by replacing the osf with a dsf.

Type 25 - Downstream hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0), obf(t_0), fc(t_0), cp(t_0)), dfc(t)) \quad (4.28)$$

An example of a Type 25 diversion structure is a gated orifice with a submerged outlet which is controlling flows from a reservoir whose depth remains relatively constant. Provision is made to observe the water flow rate downstream of the structure. The operator will adjust the gate until the osf matches the obf.

Type 26 - Downstream hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(osf(t_0, t), obf(t_0, t), fc(t_0, t), cp(t_0, t)), dfc(t)) \quad \ldots (4.29)$$

An example of a Type 26 diversion structure is an automated Type 25 structure. This may be achieved by providing the controller with the appropriate psc-function and the observed downstream flow information. The controller will compare the osf and the obf and will operate the actuator to adjust the gate opening until the osf and obf match.

Type 27 - Downstream hydraulic control
- Fixed operation
- Closed-loop control of operation

No examples of this type of diversion structure exist.

Type 28 - Downstream hydraulic control
- Fixed-automatic operation
- Closed-loop control

Flow through this type of diversion structure may be described using the following equation:

$$Q(t) = Ra(psc(dsf(t), obf(t), fc(t), cp(t)), dfc(t)) \qquad (4.30)$$

The example provided for a Type 26 diversion structure may be converted to a Type 24 structure by replacing the osf with a dsf.

Type 29 - Upstream/downstream hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(psc(osf(t_0), obf(t_0), fc(t_0), cp(t_0)), ufc(t), dfc(t)) \quad \ldots (4.31)$$

An example of a Type 29 diversion structure is a gated orifice with submerged outlet delivering water from a canal, which has a variable depth of water, into another canal, which also has a variable depth of water. Provision is made to observe the water flow downstream of the diversion structure. The operator will adjust the gate until osf matches the obf.

Type 30 - Upstream/downstream hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs(osf(t_0, t), obf(t_0, t), fc(t_0, t), cp(t_0, t), ufc(t), dfc(t)) \quad (4.32)$$

An example of a Type 30 diversion structure is an automated Type 29 structure. This may be achieved by providing the controller with the appropriate pcs-function and observed downstream flow information. The controller will compare the osf and obf and will operate the actuator to adjust the gate opening until the osf and obf match.

Type 31 - Upstream/downstream hydraulic control
- Fixed operation
- Closed-loop control of operation

No examples of this type of diversion structure exist.

Type 32 - Upstream/downstream hydraulic control
- Fixed-automatic operation
- Closed-loop control of operation

Flow through diversion structures of this type may be described by the following equation:

$$Q(t) = Ra(pcs(dsf(t), obf(t), fc(t), cp(t)), ufc(t), dfc(t)) \quad (4.33)$$

The example provided for Type 30 diversion structures may be converted to a Type 32 structure by replacing the osf with a dsf.

4.2.3.3 Complex Diversion Structures

The equations describing flow through complex diversion structures may be simply constructed by adding the equations corresponding to each outlet of the structure. For example, if a diversion structure has three outlets; two identical Type 5's and one Type 6, the equation describing the flow through the structure may be written as:

$$\begin{aligned} Q(t) = {} & [Ra(psc(osf(t_0), fc(t_0), cp(t_0)), ufc(t))]_1 \\ & + [Ra(psc(osf(t_0), fc(t_0), cp(t_0)), ufc(t))]_2 \\ & + \lfloor Ra(psc(osf(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t_0, t))\rfloor_3 \end{aligned} \quad \ldots (4.34)$$

The first two terms pertain to the Type 5 outlets and the third term to the Type 6 outlet.

4.2.4 Summary of Diversion Structure Types

The classification of diversion structures is summarized in Table 4.1. An asterisk beside the type number indicates that examples of this type of structure do not exist.

4.3 Transfer Structures

4.3.1 Water Management Function

Transfer structures are used either singly or in combination to transport water between diversion structures.

Table 4.1 Diversion Structure Classification System

Type No.		Hydraulic Control	Method of Operation	Type of Control
1		positive	manual/remote	open-loop
2		positive	automatic	open-loop
3		positive	fixed	open-loop
4		positive	fixed-automatic	open-loop
5		upstream	manual/remote	open-loop
6		upstream	automatic	open-loop
7		upstream	fixed	open-loop
8		upstream	fixed-automatic	open-loop
9		downstream	manual/remote	open-loop
10		downstream	automatic	open-loop
11		downstream	fixed	open-loop
12		downstream	fixed-automatic	open-loop
13		upstream/downstream	manual/remote	open-loop
14		upstream/downstream	automatic	open-loop
15		upstream/downstream	fixed	open-loop
16		upstream/downstream	fixed-automatic	open-loop
17		positive	manual/remote	closed-loop
18		positive	automatic	closed-loop
19	*	positive	fixed	closed-loop
20		positive	fixed-automatic	closed-loop
21		upstream	manual/remote	closed-loop
22		upstream	automatic	closed-loop
23	*	upstream	fixed	closed-loop
24		upstream	fixed-automatic	closed-loop
25		downstream	manual/remote	closed-loop
26		downstream	automatic	closed-loop
27	*	downstream	fixed	closed-loop
28		downstream	fixed-automatic	closed-loop
29		upstream/downstream	manual/remote	closed-loop
30		upstream/downstream	automatic	closed-loop
31	*	upstream/downstream	fixed	closed-loop
32		upstream/downstream	fixed-automatic	closed-loop

4.3.2 Classification System

Transfer structures may be classified as follows: canals, canal associated specialty structures, pipelines and pipeline associated specialty structures.

4.3.2.1 Canals

Canals may be classified into four principal types: earth canals, buried-membrane lined canals, surface membrane lined canals and compound canals.

4.3.2.1.1 Earth Canals

Earth canals include all canals formed entirely by soil material. These canals may be lined with clay-type soils to reduce seepage losses. Earth canals provide a continuous seepage path from inside the canal to the surrounding ground water and may be subject to erosion processes and weed infestations.

4.3.2.1.2 Buried-Membrane Lined Canals

Buried-membrane lined canals are usually constructed using a flexible impermeable membrane such as PVC plastic, butyl rubber or polyethylene plastic covered with a layer or layers of soil material. Seepage of water from these canals will not occur. However, buried-membrane lined canals may be subject to erosion processes and weed infestations.

4.3.2.1.3 Surface-Membrane Lined Canals

Surface-membrane lined canals are constructed with an impervious surface layer, such as concrete or asphalt, overlying the pervious soil material. Seepage of water from these canals will not occur. Surface-membrane lined canals are not normally subject to erosion processes or weed infestations.

4.3.2.1.4 Compound Canals

Compound canals incorporate two or more lining methods in their cross-section. Normally, a portion of the canal cross-section will be left unlined (earth-lined), and the remainder will be lined with a buried or surface membrane. The membrane is often buried in the middle of the canal to form a cut-off curtain which will further modify the path the seeping water will take.

4.3.2.2 Pipelines

There are two types of pipeline systems used to transfer water: open pipeline systems and closed pipeline systems.

4.3.2.2.1 Open Pipeline Systems

Open pipeline systems do not necessarily flow full throughout their length. Flow conditions may vary from open channel to pipeflow. In the latter case, the flow is usually low pressure.

Similar to canals, water entering an open pipeline system through a diversion structure will continue through the pipeline until it is diverted or spilled. Flow through the system is similar in many respects to that which occurs in a storm sewer.

4.3.2.2.2 Closed Pipeline Systems

Closed pipeline systems are pressurized either by sufficient emersion of the diversion structure at the pipe inlet or by a pump-motor assembly. Water flow into a closed pipeline system will occur in direct response to actual water demands made at diversion points along the pipeline system itself. Except when system flushing is required, water is not normally spilled from closed pipeline systems.

4.3.2.3 Canal-Associated Specialty Structures

Canal-associated specialty structures include those transfer components which facilitate:

1. the continuity of canals over depressions, (e.g. flumes and inverted siphons);
2. the continuity of canals under/over obstacles, (e.g. flume culverts and sag culverts);
3. the transition between two canals of different cross-section;
4. the transition between canals and other canal associated specialty structures, diversion structures, depth-control structures or open-pipeline systems;
5. a positive change in canal bed elevation (e.g. lift station);

6. a negative change in canal bed elevation (e.g. vertical drop-structure and chutes);
7. the dissipation of water energy (e.g. stilling-basins, roller buckets and flip-bucket - plunge pools);
8. water measurement (e.g. weirs and flumes);
9. trash control;
10. sediment control (e.g. sediment traps below canal inlet);
11. change in direction of flow;
12. scour control; and,
13. backwater control.

Canal-associated specialty structures are designed to minimize their interference with the flow of water in the canal itself or with the operation of diversion and depth-control structures. Ideally, the flow characteristics of a canal which incorporates several specialty structures would be similar to the flow characteristics of a canal with no such structures.

4.3.2.4 Pipeline-Associated Specialty Structures

Pipeline-associated specialty structures include those transfer components which provide for:

1. the transition between pipelines of different dimension;
2. the continuity of pipelines, over depressions (open pipeline systems);
3. the continuity of pipelines under/over obstacles (open pipeline systems);

4. a positive change in elevation of pipeline invert (open pipeline systems);
5. a negative change in elevation of pipeline invert (open pipeline systems);
6. the release of trapped or entrained air;
7. the entry of air into pipeline;
8. water measurement;
9. trash control;
10. sediment control;
11. surge protection;
12. prevention of flow reversal (closed pipeline systems); and,
13. change in direction of flow.

Similar to canal-associated specialty structures, pipeline-associated specialty structures are designed to minimize their interference with the flow of water in the pipeline itself or with the operation of diversion and depth-control structures.

4.3.3 General Equations Describing the Operation of Water Transfer Structures

Transfer structures are not normally considered operable; that is, there is either no provision for adjustment to take place or all structural elements are fixed in position. However, the operator must be aware of the effect the transfer structure will have on flow conditions upstream or downstream of the transfer structure and within the transfer structure itself in the case of canals and pipelines.

4.3.3.1 Canals and Open-Pipelines

Canals and open-pipelines may contain considerable water storage and the flow rate along their length can vary substantially at any particular time. This variability can be further exacerbated by distributed water inflows and outflows and by time-dependent canal or pipeline physical characteristics.

When using canals or open pipelines the operator is concerned with estimating or observing the magnitude of steady-state flow conditions and the time the steady-state flow conditions will be achieved. He will use this information to operate diversion and depth-control structures. If the diversion and depth-control structures are automated, the controller has similar considerations.

An equation which may be used to qualitatively describe the flow through an individual cross-section of a canal or open-pipeline, located a distance x downstream from the entrance to the reach, is as follows:

$$Q(x, t) = f(pcs, Q_L(t), ufc(t), dfc(t), x) \tag{4.35}$$

where: psc describes the physical characteristics of the canal or open pipeline which determines its hydraulic properties. In many instances, pcs can vary with time.

$Q_L(t)$ is the distributed lateral inflow or outflow at time, t.

ufc(t) is the flow condition at the entrance to the reach at time, t.

dfc(t) is the flow condition at the end of the reach at time, t.

x is the location along the reach as measured downstream from the entrance.

$Q(x, t)$ is the flow rate at time, t, through the cross-section located a distance x downstream from reach entrance.

A detailed treatment of the theory which describes unsteady and steady flow through open channels is presented in Appendix A and Appendix B respectively.

4.3.3.2 Closed Pipelines

By definition closed pipelines operate completely full. Because of this, storage changes within closed pipelines may be assumed negligible and flow rates throughout individual reaches constant. It follows that the flow rate through a reach of a closed pipeline is completely dependent on its physical characteristics and the flow conditions at its entrance and exit. This may be restated in equation form as follows:

$$Q(t) = f(pcs, ufc(t), dfc(t)) \qquad (4.36)$$

where: $Q(t)$ is the flow rate through the pipeline at time, t.

pcs describes the physical characteristics of the pipeline which determine its hydraulic properties.

$ufc(t)$ is the flow condition at the entrance to the reach.

$dfc(t)$ is the flow condition at the end of the reach.

4.3.3.3 Canal- and Pipeline-Associated Specialty Structures

Most of the canal- and pipeline-associated specialty structures may be described in a similar manner to that used to describe fixed operation diversion and depth-control structures. This is because there is usually negligible or no water storage within the structures themselves and flow through the inlet and outlet of the structures may be assumed equal at all times. If properly designed, canal and pipeline associated structures are essentially passive in that they do not require operator control or require frequent maintenance. Their hydraulic characteristics remain constant with time.

Equations which may be used to describe flow through or past canal- and pipeline-associated specialty structures include the following:

$$Q(t) = f(pcs, ufc(t), sv(t)) \quad (4.37(a))$$

$$Q(t) = f(pcs, dfc(t), sv(t)) \quad (4.37(b))$$

$$Q(t) = f(pcs, ufc(t), dfc(t), sv(t)) \quad (4.37(c))$$

and

$$Q(t) = f(pcs, sv(t)) \quad (4.37(d))$$

where: pcs is the physical characteristic of the structure which determines its hydraulic properties. For certain structures, pcs may vary with frequency of maintenance.

ufc(t) is the upstream flow condition at time, t, which governs the flow rate through the structure.

dfc(t) is the downstream flow condition at time, t, which may govern the flow rate through the structure.

Q(t) is the flow rate through the structure at time, t.

sv(t) is the state variable at time, t, which may govern the flow rate through the structure.

4.3.4 Summary of Transfer Structure Types

The classification of transfer structure types is summarized in Table 4.2

Table 4.2 Transfer Structure Classification System

1. Canals - earth canals
   - buried membrane lined canals
   - surface membrane lined canals
   - compound canals
   - canal associated specialty structures

2. Pipelines - open pipelines
   - closed pipelines
   - pipeline associated specialty structures

4.4 Depth-Control Structures

4.4.1 Water Management Function

Depth-control structures are used to control water depth in canals, storage facilities and the head in open-pipeline systems. The main purpose of this control is to facilitate water diversion activities. Closed-pipeline systems do not normally employ devices equivalent to depth-control structures.

4.4.2 Classification System

Depth-control structures may be classified in the same manner as is used to classify diversion structures, (see Section 4.2). That is, depth-control structures are classified according to the type of hydraulic control which governs the flow of water through the structure, the method used to operate the structure, the type of control incorporated in the method used to operate the structure and the degree with which the operation of the individual diversion structure is co-ordinated with the operation of other structures, flow conditions or water demands within the conveyance system.

4.4.2.1 Hydraulic Control

Three types of hydraulic control are identified: upstream control, upstream/downstream control and positive control. Upstream control is usually associated with weirs and free-outlet type orifices; upstream/downstream control is usually associated with orifices with submerged outlets; and, positive control is usually associated with pump-motor assemblies.

4.4.2.1.1 Upstream

This type of hydraulic control implies that the flow of water through the depth-control structure is only dependent on flow conditions upstream of the structure. Most depth-control structures employing upstream hydraulic control resemble weirs or free-outlet type orifices.

4.4.2.1.2 Upstream/Downstream

Upstream/downstream hydraulic control implies that the flow of water through the depth-control structure is dependent on flow conditions upstream and downstream of the structure. The most common depth-control structure employing upstream/downstream hydraulic control is the submerged-outlet type orifice.

4.4.2.1.3 Positive

This type of hydraulic control implies that the hydraulics governing the flow of water past the depth-control structure is unaffected by flow conditions in the conveyance system itself. Depth-control structures employing positive hydraulic control are normally used in association with smaller storage facilities and are frequently used in conjunction with canals and pipelines.

4.4.2.2 Method of Operation

Depth-control structures may be classified according to method of operation; that is, according to the operability of the structure as determined by the extent of operator participation permitted in the selection of water depth upstream of the structure; and according to the operator's ability to physically adjust the structure. Four different methods of operation are identified; manual/remote, automatic, fixed and fixed-automatic.

4.4.2.2.1 Manual/Remote

Manual/remote operation of a depth-control structure implies that there is direct operator participation in selecting the desired water depth upstream of the structure and in the physical adjustment of the structure. Manual/remote operation differ only to the extent that operator presence at the structure is not required for remote operation. Remote interrogation and/or reporting of flow conditions in the vicinity of the depth-control structures or the physical status of the depth-control structure should not be confused with remote operations.

4.4.2.2.2 Automatic

Automatic operation of a depth-control structure implies that there is direct operator participation, on site or remote, in selecting the water depth upstream of the structure; but, the physical adjustment of the structure to actually achieve the desired upstream depth is performed by a controller-actuator system incorporated into the structure itself. (See discussion on automatically operated diversion structures in Section 4.2.2.2.2).

4.4.2.2.3 Fixed

Fixed method of operation depth-control structures are not operable. No operator participation is possible either in selecting the desired upstream depth or in the physical adjustment of the structure, not only because there is no provision for adjustments to take place; but also, because all structural elements are fixed in position. The hydraulic characteristics of the depth-control structure are completely established at the time construction of the structure is completed. Flow through the structure is totally dependent on the type of hydraulic control governing the flow through the structure and the flow conditions occurring during the time the structure is commissioned.

4.4.2.2.4 Fixed-Automatic

Fixed-automatic method of operation depth-control structures are also not operable. No operator participation is possible either in selecting the flow through the structure or in the physical adjustment of the structure itself. Structure elements are not fixed; however, and the structure is automatically controlled. The selection of desired upstream water depths and mode of control have been made and fixed at the time the structure was commissioned.

4.4.2.3 Control of Operation

Depth-control structures may be classified according to the type of control incorporated into the particular method used to operate the structure. Two types of control are identified; open-loop and closed-loop.

4.4.2.3.1 Open-Loop

Open-loop control implies that the depth of flow upstream of the structure is determined by the relative positioning of appropriate structural elements within the diversion structure without requiring direct observations of the upstream water depth resulting from the structure operation. Flow rate information and position of structural elements may be used in a rating function which permits prediction of water depth upstream of the depth-control structure.

4.4.2.3.2 Closed-Loop

Closed-loop control implies that depth of water upstream of the diversion structure is determined using water depth measurements made upstream of the structure being operated. The depth-control structure is adjusted until the observed depth matches the selected depth.

4.4.2.4 Co-ordination of Operation

Individual depth-control structures may be further classified according to how their operation is co-ordinated with the operation of other structures or water demands within the conveyance system. As with diversion structures, three levels are identified; system, local and independent. Discussion of each level may be found in Sections 4.2.2.4.1, 4.2.2.4.2 and 4.2.2.4.3 respectively.

4.4.2.5 Complex Depth-Control Structures

Complex depth-control structures may consist of several independently functioning outlets. Each outlet is equivalent to a single depth-control structure and may be classified and described accordingly. Several outlets, corresponding to the same classification or to several different classifications of depth-control structure may be incorporated into a single complex structure. All outlets, however, must discharge into the same canal.

4.4.3 General Equations Describing the Operation of Depth-Control Structures

General equations are presented which describe the operation of depth-control structures identified in the preceding sections. The equations are grouped according to the method of control used to operate the structures. Co-ordination of depth-control structure operation is not considered.

4.4.3.1 Open-Loop Controlled Depth-Control Structures

General equations describing the operation of open-loop controlled depth-control structures are presented as follows.

Type 1 - Upstream hydraulic control
- Manual/remote operation
- Open-loop control of operations

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0), fc(t_0), cp(t_0)), ufc(t)) \qquad (4.38)$$

where: $osh_u(t_0)$ is the operator selected depth of water upstream of the structure at the time of operation, t_0.

$fc(t_0)$ is the flow condition which is considered at the time the structure is operated. More than one fc may be considered.

$cp(t_0)$ is the structure control parameter at the time the structure is operated. More than one cp may be included.

psc( ) is the adjustable physical characteristic of the structure affecting its hydraulic control. Normally, there is only one pcs function; though, several are possible.

ufc(t) is the upstream flow condition which governs flow through the structure. More than one ufc may be considered.

Ra( ) is the rating function for the structure. The rating function permits the calculation of actual water flow through the structure, Q(t). (See discussion of Type 1 diversion structure in Section 4.2.3.1).

t_o is the actual time the structure is operated.

t is the actual time; not necessarily since the time the structure was operated.

Q(t) is the actual water flow rate through the structure at time, t.

An example of a Type 1(a) depth-control structure is a simple check-structure illustrated in Figure 4.1(a). The weir height is adjusted by adding or subtracting stop-logs. Obviously, this structure may only be operated manually. The ufc which is governing the flow through the weir is upstream water depth, though upstream water velocity may also be significant. One of the important control parameters is the size of the possible increments of adjustment; that is, the depth of the individual stop logs.

An example of a Type 1(b) depth-control structure is a manually operated radial gate installed immediately upstream of a chute as is illustrated in Figure 4.1(b). The ufc considered would be the upstream depth of flow, and possibly, upstream flow velocity.

In order to qualify for the open-loop classification, both Type 1(a) and Type 1(b) depth-control structures would use observed, estimated or anticipated upstream flow rate when determining the correct adjustment; that is, when determining the pcs. It is also likely that the depth of water immediately upstream of the structure at the time of operation would also be used.

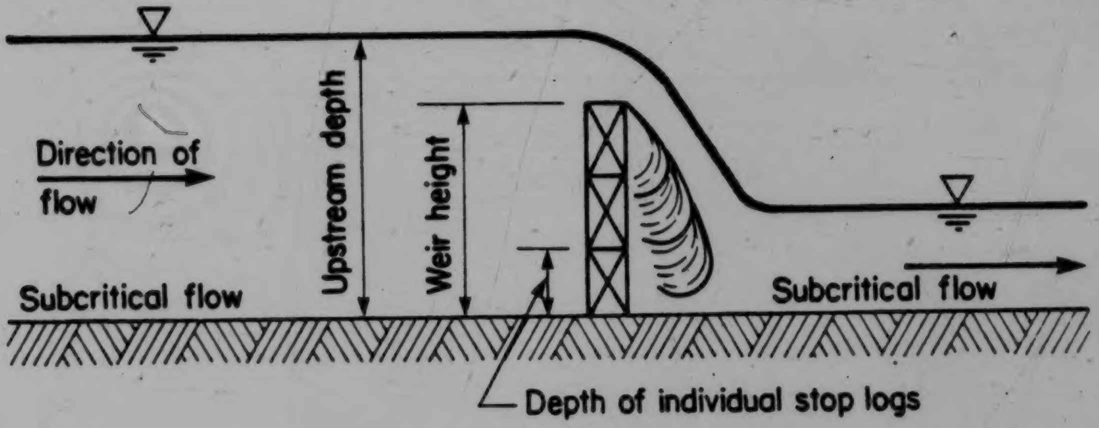

Direction of flow
Upstream depth
Weir height
Subcritical flow
Subcritical flow
Depth of individual stop logs

Figure 4.1(a) Type 1(a) depth-control structure.

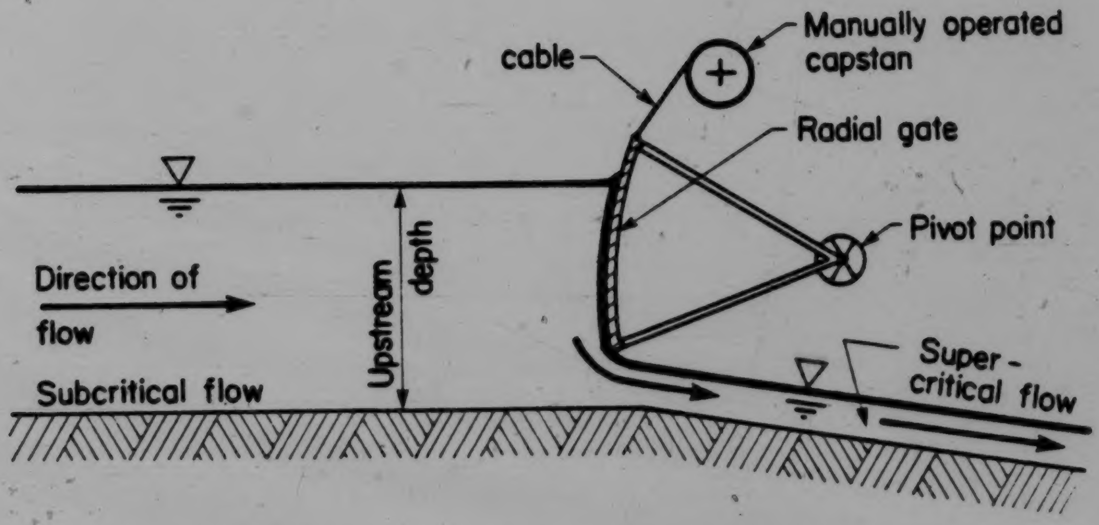

cable
Manually operated capstan
Radial gate
Pivot point
Direction of flow
Upstream depth
Subcritical flow
Super-critical flow

Figure 4.1(b) Type 1(b) depth-control structure.

Type 2 - Upstream hydraulic control
- Automatic operation
- Open-loop control of operation

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_o, t), fc(t_o, t), cp(t_o, t)), ufc(t)) \quad (4.39)$$

where: $osh_u(t_o)$ is the operator selected depth upstream of the structure supplied to the controller at time of operation, t_o; and, possibly modified by it anytime thereafter according to algorithms fully understood and controlled by the operator.

$fc(t_o, t)$ is the flow condition which may be considered at the time the depth upstream of the structure is selected; and, the flow condition considered anytime thereafter by the controller. More than one flow condition may be considered.

$cp(t_o, t)$ is the structure control parameter considered at the time the depth upstream of the structure is selected and anytime thereafter by the controller. More than one structure control parameter may be considered.

An example of a Type 2(a) depth-control structure is the automated pivoted weir-type check structure illustrated in Figure 4.2(a). A flow condition which may be considered by the operator and controller is the observed discharge some distance upstream of the structure, (e.g. below an upstream diversion structure). This would be represented as

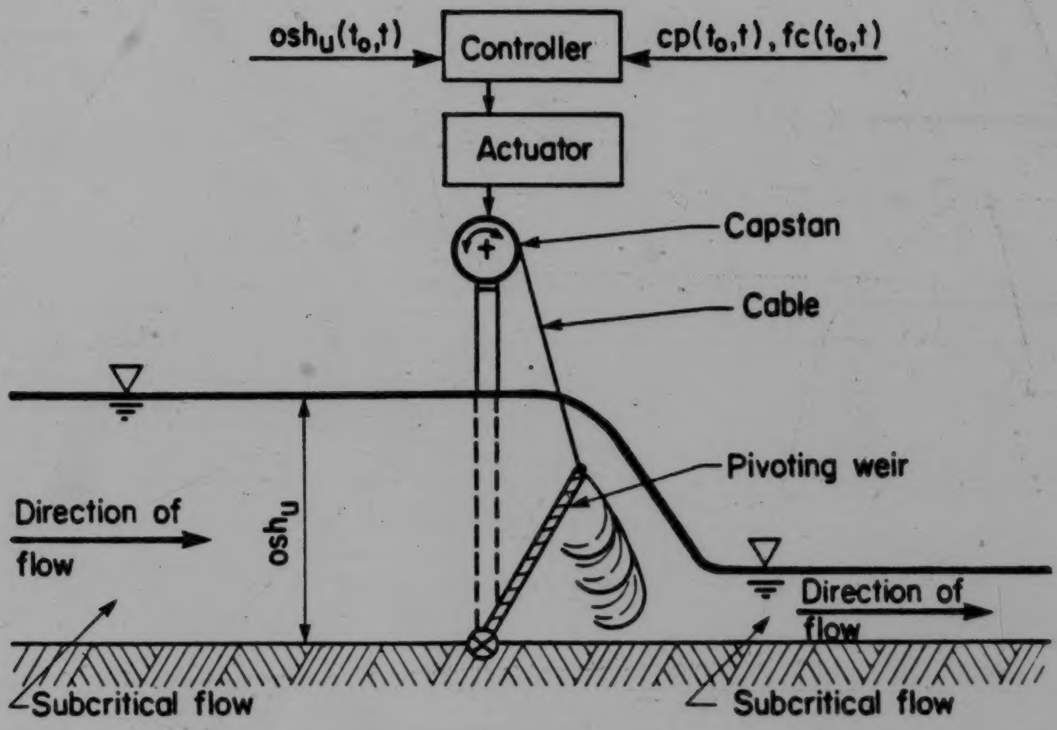

$osh_u(t_o,t)$
Controller
$cp(t_o,t), fc(t_o,t)$
Actuator
Capstan
Cable
Pivoting weir
Direction of flow
osh_u
Direction of flow
Subcritical flow
Subcritical flow

Figure 4.2(a) Type 2(a) depth-control structure.

$fc(t_0, t)$. Control parameters which may be considered by the operator and controller are the actual position of the pivoted weir itself and the rate of adjustment permitted the controller. The controller would operate the actuator in order to adjust the gate opening to maintain the desired osh_u.

An example of a Type 2(b) depth-control structure is the automated radial gate type check structure illustrated in Figure 4.2(b). This structure is operated in precisely the same manner as the example provided for Type 2(a) depth-control structures.

Type 3 - Upstream hydraulic control
- Fixed operation
- Open-loop control of operation

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the following equation:

$$Q(t) = Ra(pcs, ufc(t)) \tag{4.40}$$

All physical characteristics of the structure are fixed.

Examples of Type 3(a) and Type 3(b) are not common. This is especially true for Type 3(b). Type 3(a) structures are usually used as part of larger complex structure where the fixed weir portions of the entire structure may be quite large compared to the adjustable portions of the structure.

Type 3 depth-control structures should not be confused with Type 7 diversion structures.

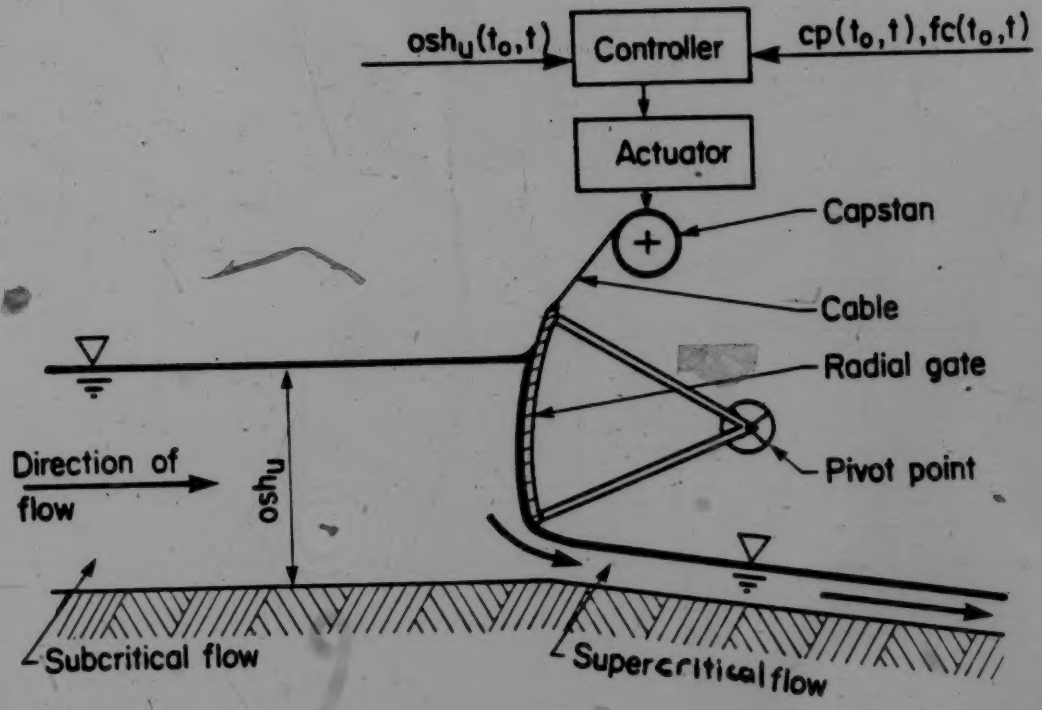

$osh_u(t_o,t)$
Controller
$cp(t_o,t), fc(t_o,t)$
Actuator
Capstan
Cable
Radial gate
Pivot point
Direction of flow
osh_u
Subcritical flow
Supercritical flow

Figure 4.2(b) Type 2(b) depth-control structure.

Type 4 - Upstream hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the equation:

$$Q(t) = Ra(pcs(dsh_u(t),\ fc(t),\ cp(t)),\ ufc(t)) \tag{4.41}$$

where: $dsh_u(t)$ is the designer selected upstream depth. This quantity may be fixed at the time of construction or may vary with time according to some pre-established algorithm.

$cp(t)$ is the structure control parameter which may be fixed at the time of construction or may vary with time according to some pre-established algorithm. More than one cp may be considered.

$fc(t)$ is the flow condition which is considered by the controller prior to structure operation. More than one fc may be considered.

Examples of Type 4(a) and (b) depth-control structures are Type 2(a) and (b) structures with $osh_u(t_o)$ and $cp(t_o, t)$ replaced with $dsh_u(t)$ and $cp(t)$ respectively. Type 4(a) and (b) depth-control structures have limited applications in irrigation conveyance systems.

Type 5 - Upstream/downstream hydraulic control
- Manual/remote operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described

using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0), fc(t_0), cp(t_0)), ufc(t), dfc(t)) \quad (4.42)$$

where: dfc(t) is the downstream flow condition at anytime t which also governs flow through the structure. More than one dfc may be considered.

An example of a Type 5 depth-control structure is the radial gate shown in Figure 4.3. Flow conditions which may be considered at time of operation include; observed upstream depth of water, observed downstream depth of water and observed, estimated or anticipated flow rate upstream of structure.

Type 6 - Upstream/downstream hydraulic control
- Automatic operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t), dfc(t)) \quad \ldots (4.43)$$

Examples of Type 6 depth-control structures are uncommon. Normally, when Type 5 structures are automated, control would be closed-loop (see Type 18 depth-control structure). However, if flow conditions considered included information such as anticipated flow rate immediately upstream of the structure or the observed or estimated flow rate below a diversion structure some distance upstream of the depth-control structure, open-loop control would exist.

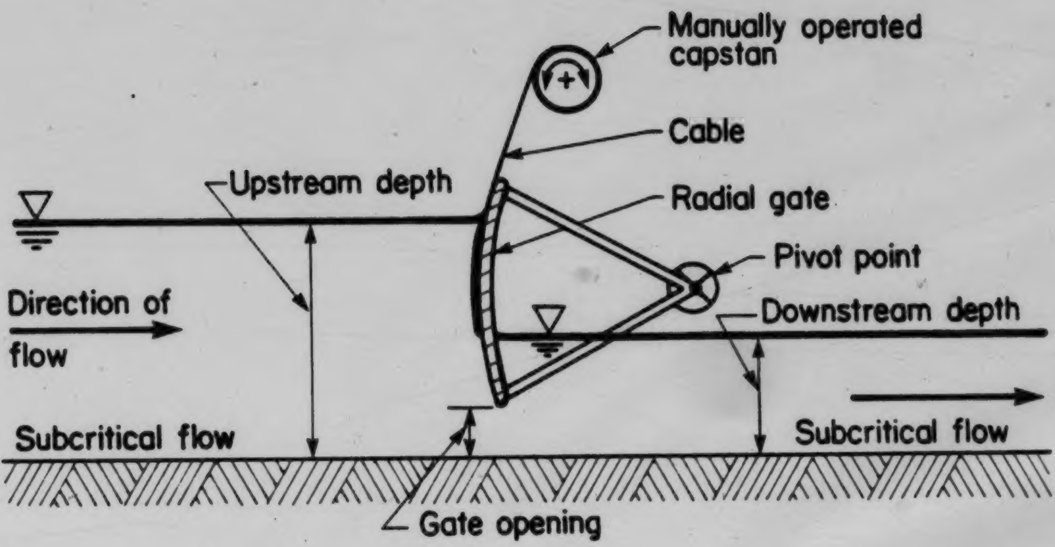

Manually operated capstan
Cable
Upstream depth
Radial gate
Pivot point
Direction of flow
Downstream depth
Subcritical flow
Subcritical flow
Gate opening

Figure 4.3 Type 5 depth-control structure.

Type 7 - Upstream/downstream hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through depth-control structures of this type may be described using the following equation:

$$Q(t) = Ra(pcs, ufc(t), dfc(t)) \tag{4.44}$$

Examples of Type 7 depth-control structures are readily conceptualized but have limited application in irrigation conveyance systems. Type 7 depth-control structures should not be confused with Type 15 diversion structures.

Type 8 - Upstream/downstream hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

Flow through this type of depth-control structure is described by the equation:

$$Q(t) = Ra(pcs(dsh_u(t), fc(t), cp(t)), ufc(t), dfc(t)) \tag{4.45}$$

An example of a Type 8 depth-control structure is a Type 6 structure with $osh_u(t_0, t)$ and $cp(t_0, t)$ replaced with $dsh_u(t)$ and $cp(t)$ respectively. Type 8 depth-control structures have limited application in irrigation conveyance systems.

Type 9 - Positive hydraulic control
- Manual/remote operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t), sv(t_0), fc(t_0), cp(t_0)), sv(t)) \qquad (4.46)$$

where: $sv(t_0)$ is the state variable which will determine the hydraulic characteristics of the structure considered by the operator at time of operation, t_0. There may be more than one $sv(t_0)$.

$sv(t)$ is the state variable which determines the hydraulic characteristics of the structure at any time, t. There may be more than one continually varying state variable.

Note that the state variables considered by the operator will not necessarily include all of the state variables determining flow through the structure.

Type 9 depth-control structures have a limited application in irrigation conveyance systems. If positive hydraulic controlled depth-control structures are to be used, they would likely use closed-loop control; that is, they would be Type 21 depth-control structures.

Type 9 depth-control structures should not be confused with lift-stations which are really a form of transfer structure.

Type 10 - Positive hydraulic control
- Automatic operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0, t), sv(t_0, t), fc(t_0, t), cp(t_0, t)), sv(t)) \qquad \ldots (4.47)$$

where: $sv(t_0, t)$ is the state variable which may be considered by the operator at the time the flow rate through the structure is selected and the state variable considered any time thereafter by the controller. More than one $sv(t_0, t)$ may be considered. However, the state variables considered by the operator and the controller will not necessarily include all of the state variables determining flow through the structure.

Type 10 depth-control structures have a limited application in irrigation conveyance systems.

Type 11 - Positive hydraulic control
- Fixed operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs, sv(t)) \tag{4.48}$$

Type 11 depth-control structures have a limited application in irrigation conveyance systems.

Type 12 - Positive hydraulic control
- Fixed-automatic operation
- Open-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsh_u(t), sv(t), fc(t), cp(t)), sv(t)) \tag{4.49}$$

Type 12 depth-control structures have limited application in irrigation conveyance systems.

4.4.3.2 Closed-Loop Controlled Depth-Control Structures

General equations describing the operation of closed-loop controlled depth-control structures are presented as follows:

Type 13 - Upstream hydraulic control
- Manual/remote operation
- Closed-loop control of operation

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_o), obh_u(t_o), cp(t_o)), ufc(t)) \qquad (4.50)$$

where: $obh(t_o)$ is the observed depth of water upstream of the depth-control structure at time of operation, t_o.

An example of a Type 13(a) depth-control structure is the manually operated weir-type check illustrated in Figure 4.4(a). Weir-height is adjusted by adding or subtracting stop logs until the observed depth of water upstream of the structure, $obh_u(t_o)$, is considered sufficiently close to the depth the operator has selected, $osh_u(t_o)$.

An example of a Type 13(b) depth-control structure is the manually operated radial gate illustrated in Figure 4.4(b). The operator will adjust the opening of the gate until the observed depth of the water upstream of the structure, $obh_u(t_o)$, matches the selected depth of water, $osh_u(t_o)$.

The manual/remote operation of Type 13 depth-control structures can be very time consuming for the operator. Each time an adjustment of the

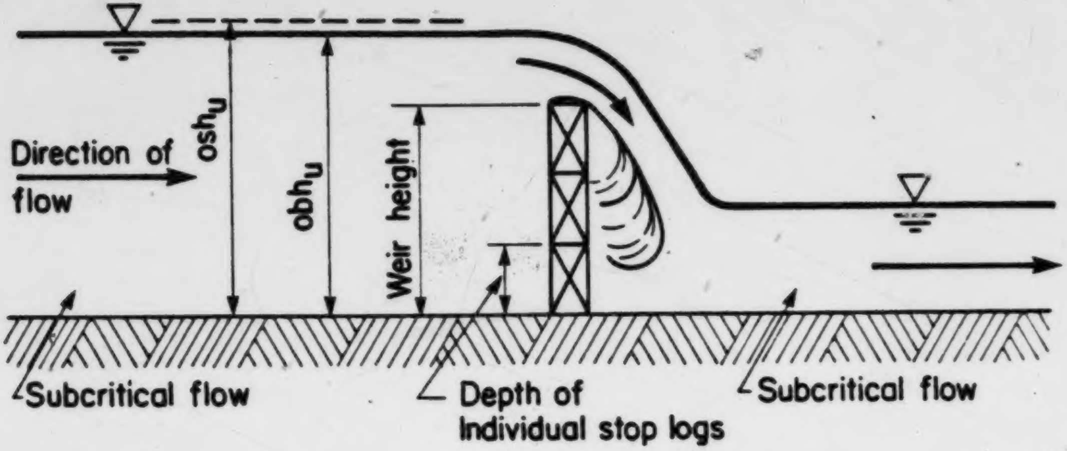

Direction of flow
osh_u
obh_u
Weir height
Subcritical flow
Depth of Individual stop logs
Subcritical flow

Figure 4.4(a) Type 13(a) depth-control structure.

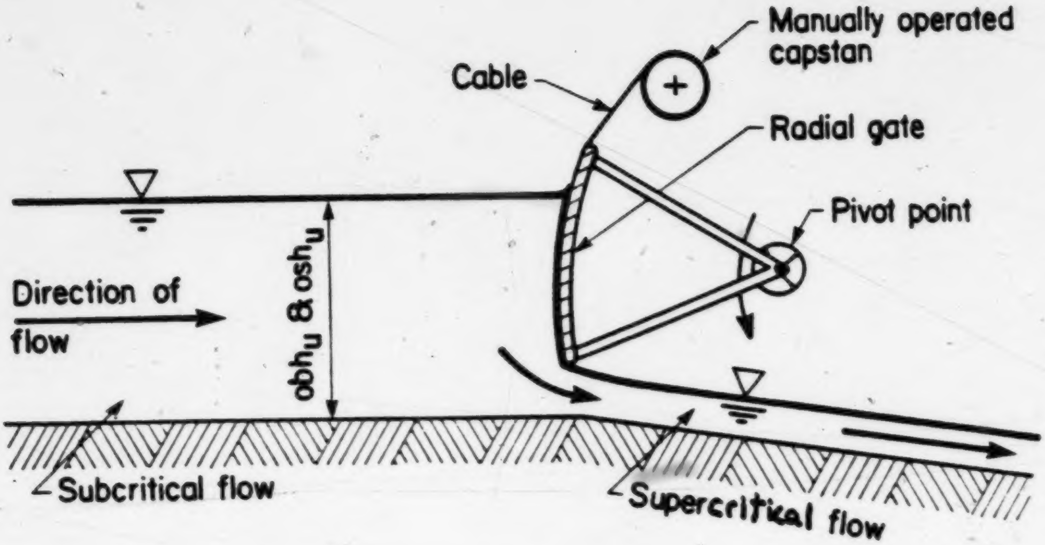

Manually operated capstan
Cable
Radial gate
Pivot point
Direction of flow
obh_u & osh_u
Subcritical flow
Supercritical flow

Figure 4.4(b) Type 13(b) depth-control structure.

structure is made, he must expect to wait until steady-state conditions have been approximated before evaluating the affect his adjustments have had on water depth. This situation may be aggravated if upstream discharge is also varying. In order to avoid excessive operation time requirements, the operator will frequently not attempt to perform the adjustments unless he is sure the flow has stabilized or the operator may attempt to estimate the flow which the structure must pass and operate it using open-loop control; that is, as a Type 1 structure. Another solution to avoiding time consuming operations is automation, that is, using Type 14 depth-control structures.

Type 14 - Upstream hydraulic control
- Automatic operation
- Closed-loop control of operation

(a) weir-type

(b) free-outlet orifice-type

Flow through these types of depth-control structures may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_o, t), obh_u(t_o, t), cp(t_o, t)), ufc(t)) \quad (4.51)$$

where: $obh_u(t_o, t)$ is the observed depth of water upstream of the depth-control structure which may be considered by the operator at the time the depth upstream of the structure is selected, t_o, and any time thereafter by the controller.

An example of a Type 14(a) depth-control structure is an automated pivoted-weir type check structure illustrated in Figure 4.5(a).

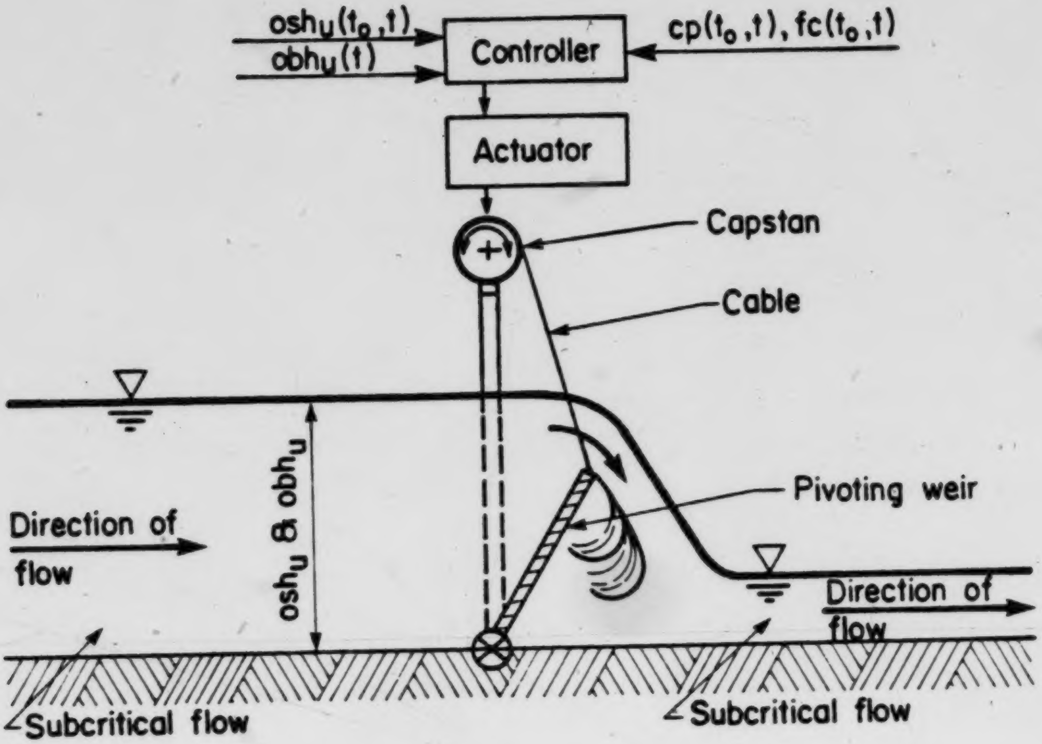

$osh_u(t_o,t)$
$obh_u(t)$
Controller
$cp(t_o,t), fc(t_o,t)$
Actuator
Capstan
Cable
Pivoting weir
Direction of flow
osh_u & obh_u
Direction of flow
Subcritical flow
Subcritical flow

Figure 4.5(a) Type 14(a) depth-control structure.

Weir-height is adjusted by the controller until $obh_u(t_o, t)$ matches $osh_u(t_o)$.

An example of a Type 14(b) depth-control structure is an automated radial-gate type check structure illustrated in Figure 4.5(b). Gate opening is adjusted by the controller until $obh_u(t_o, t)$ matches $osh_u(t_o,t)$.

Type 15 - Upstream hydraulic control
- Fixed operation
- Closed-loop control of operation

Examples of Type 15 depth-control structures do not exist.

Type 16 - Upstream-hydraulic control
- Fixed-automatic operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsh_u(t), obh_u(t), cp(t)), ufc(t)) \tag{4.52}$$

where: $obh_u(t)$ is the observed depth of water upstream of the structure at time, t.

Examples of Type 16 depth-control structures may be developed by replacing the $osh_u(t_o, t)$ and $cp(t_o, t)$ characteristics in Type 14 structures with $dsh_u(t)$ and $cp(t)$ characteristics. Type 16 depth-control structures have limited application in irrigation conveyance systems.

Type 17 - Upstream/downstream hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described

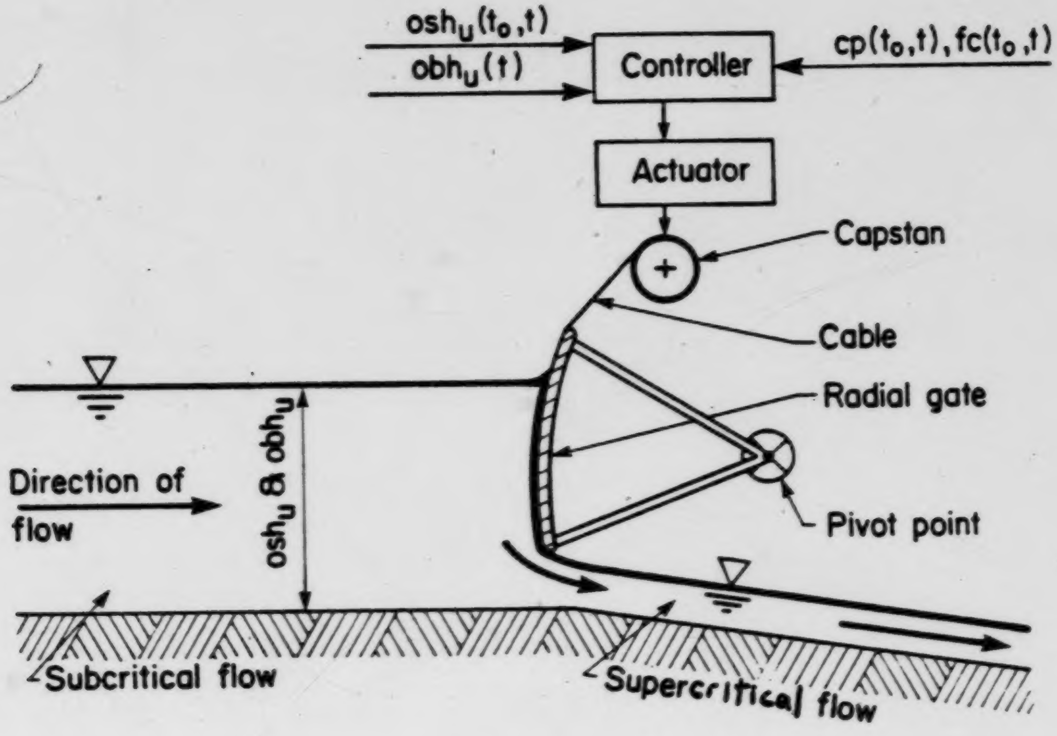

$osh_u(t_o,t)$
$obh_u(t)$
Controller
$cp(t_o,t), fc(t_o,t)$
Actuator
Capstan
Cable
Radial gate
osh_u & obh_u
Direction of flow
Pivot point
Subcritical flow
Supercritical flow

Figure 4.5(b) Type 14(b) depth-control structure.

using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_o), obh_u(t_o), cp(t_o)), ufc(t), dfc(t)) \quad (4.53)$$

An example of a Type 17 depth-control structure is the radial gate shown in Figure 4.6. Gate opening is adjusted until $osh_u(t_o)$ matches $obh_u(t_o)$.

Similar to Type 13 depth-control structures, operation of Type 17 structures can be very time consuming for the operator.

Type 18 - Upstream/downstream hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through this type of depth-control structure is described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_o, t), obh_u(t_o, t), cp(t_o, t)), ufc(t), dfc(t)) \quad \ldots (4.54)$$

An example of a Type 18 depth-control structure is the automated radial gate illustrated in Figure 4.7. Gate opening is adjusted by the controller until $obh_u(t_o, t)$ matches $osh(t_o, t)$.

Type 19 - Upstream/downstream hydraulic control
- Fixed operation
- Closed-loop control of operation

Examples of Type 19 depth-control structures do not exist.

Type 20 - Upstream/downstream hydraulic control
- Fixed-automatic operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described

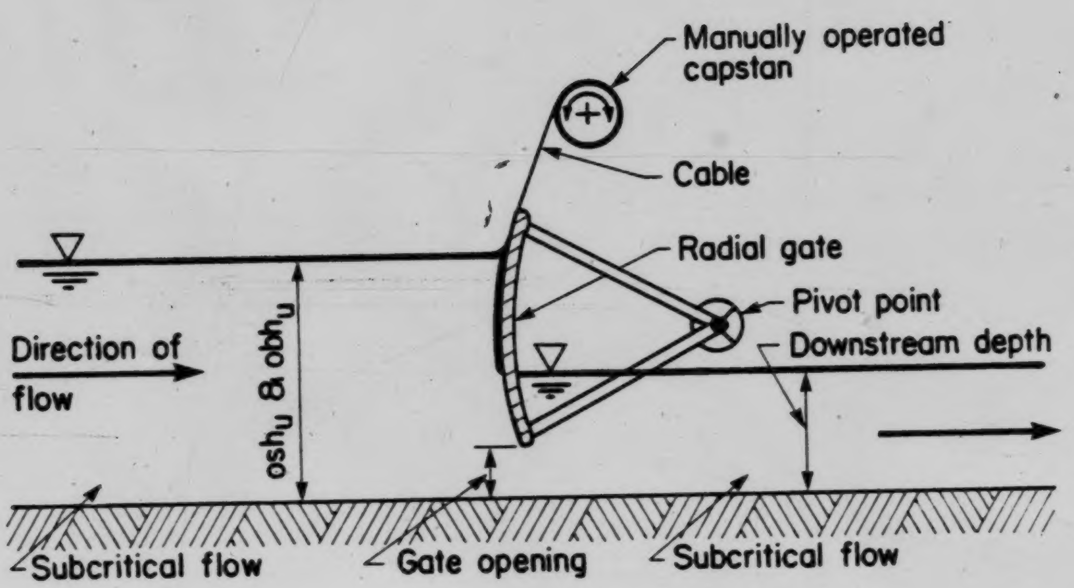

Manually operated capstan
Cable
Radial gate
Pivot point
Direction of flow
Downstream depth
osh_u & obh_u
Subcritical flow
Gate opening
Subcritical flow

Figure 4.6 Type 17 depth-control structures.

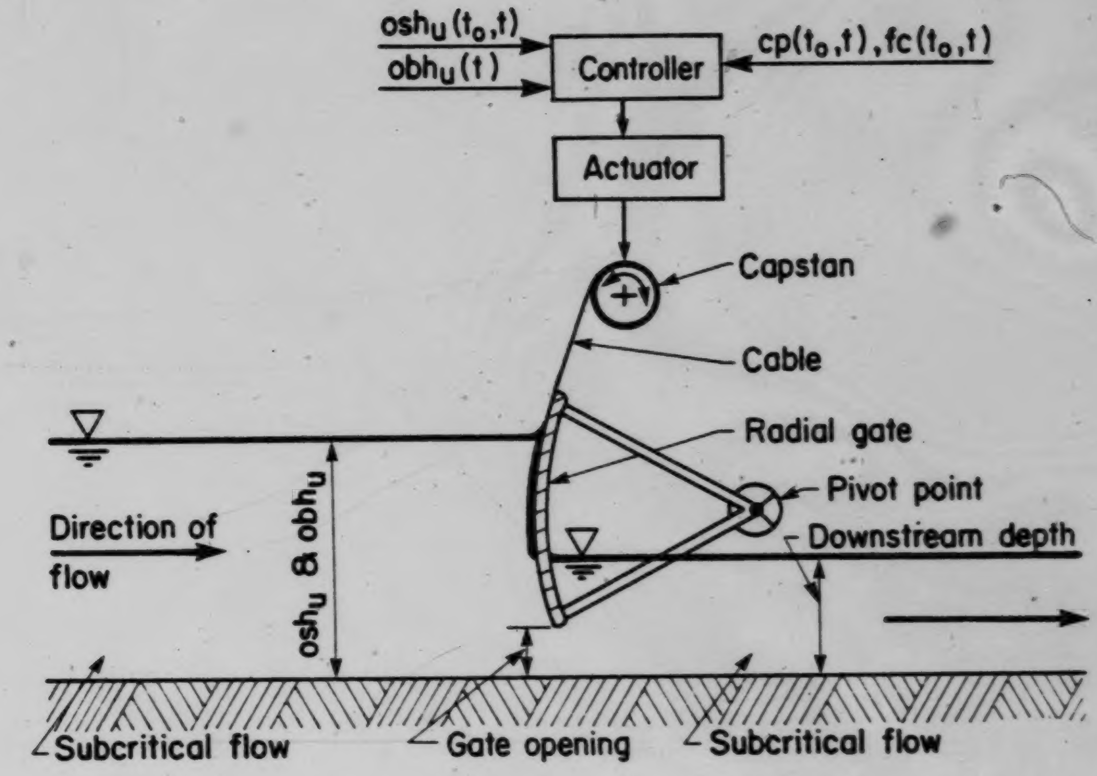

$osh_u(t_o,t)$
$obh_u(t)$
Controller
$cp(t_o,t), fc(t_o,t)$
Actuator
Capstan
Cable
Radial gate
Pivot point
Downstream depth
Direction of flow
osh_u & obh_u
Subcritical flow
Gate opening
Subcritical flow

Figure 4.7 Type 18 depth-control structure.

using the following equation:

$$Q(t) = Ra(pcs(dsh_u(t), obh_u(t), cp(t)), ufc(t), dfc(t)) \quad (4.55)$$

An example of a Type 20 depth-control structure is a Type 18 with $osh(t_0, t)$ and $cp(t_0, t)$ replaced with $dsh_u(t_0)$ and $cp(t)$. Type 20 structures have limited application in irrigation conveyance systems.

Type 21 - Positive hydraulic control
- Manual/remote operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0), obh_u(t_0), cp(t_0), sv(t_0)), sv(t)) \quad (4.56)$$

An example of a Type 21 depth-control structure, also operating as a lift-station, is shown in Figure 4.8. (This particular structure could also be called a multifunction component).

Type 22 - Positive hydraulic control
- Automatic operation
- Closed-loop control of operation

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(osh_u(t_0, t), obh_u(t_0, t), cp(t_0, t), sv(t_0, t)), sv(t)) \quad (4.57)$$

An example of a Type 22 depth-control structure is illustrated in Figure 4.9. The controller would adjust pump speed in order to match $obh_u(t_0, t)$ with $osh_u(t_0, t)$.

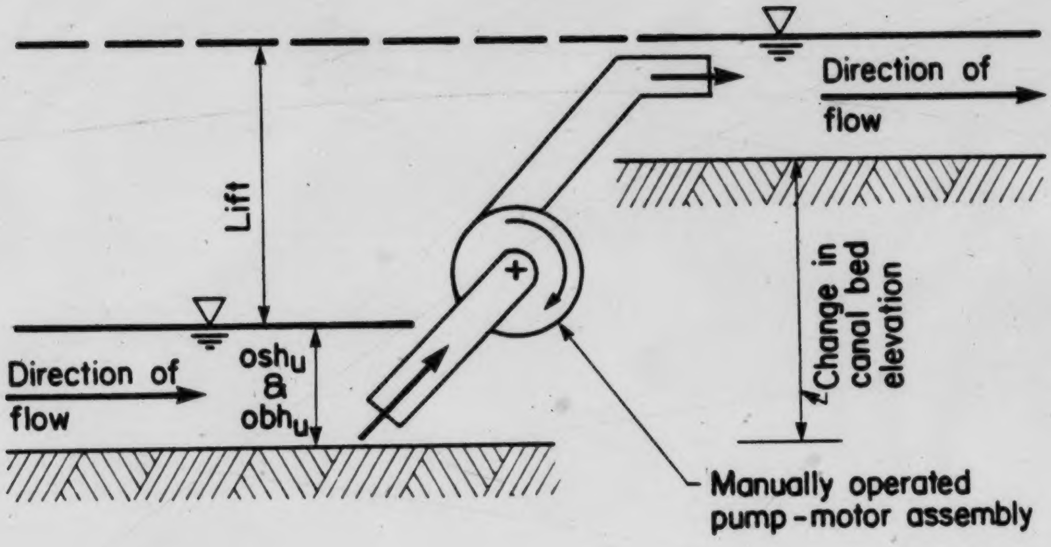

Direction of flow
Lift
Direction of flow
osh_u & obh_u
Change in canal bed elevation
Manually operated pump-motor assembly

Figure 4.8 Type 21 depth-control structure.

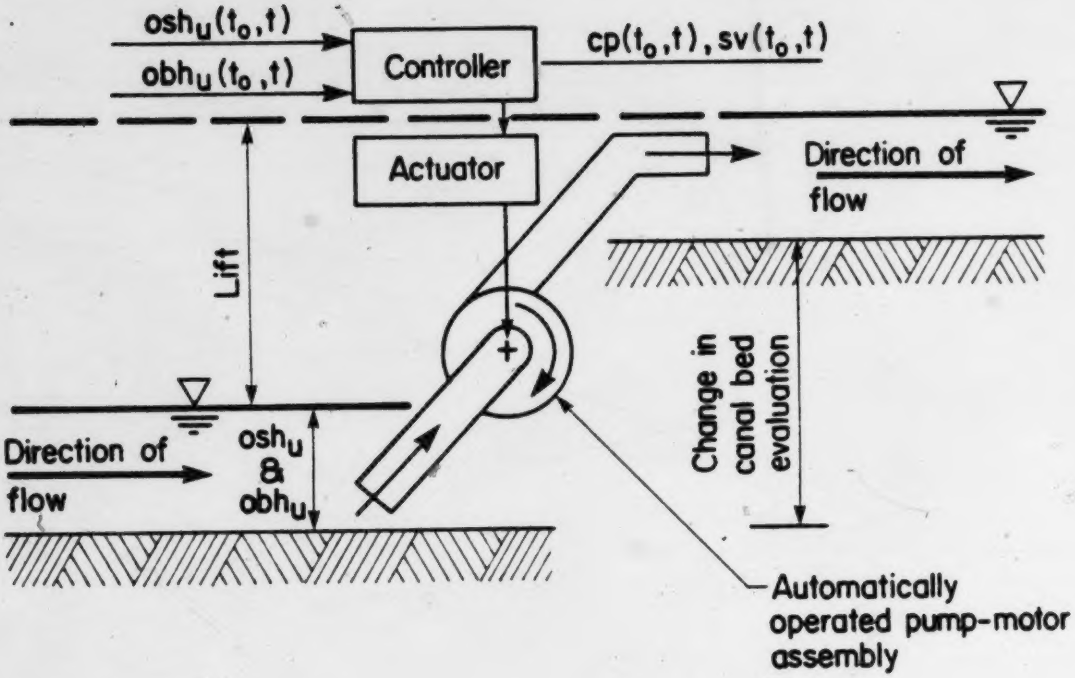

$osh_u(t_o,t)$
$obh_u(t_o,t)$
Controller
$cp(t_o,t), sv(t_o,t)$
Actuator
Direction of flow
Lift
Direction of flow
osh_u & obh_u
Change in canal bed evaluation
Automatically operated pump-motor assembly

Figure 4.9 Type 22 depth-control structure.

Type 23 - Positive hydraulic control
- Fixed operation
- Closed-loop control of operation

Examples of Type 23 depth-control structures do not exist.

Type 24 - Positive hydraulic control
- Fixed-automatic operation
- Closed-loop control

Flow through this type of depth-control structure may be described using the following equation:

$$Q(t) = Ra(pcs(dsh_u(t), obh_u(t), cp(t), sv(t)), sv(t)) \tag{4.58}$$

An example of a Type 24 depth-control structure is the Type 22 example illustrated in Figure 4.9 with the exception that $osh_u(t_0, t)$, $cp(t_0, t)$ and $sv(t_0, t)$ are replaced with $osh_u(t)$, $cp(t)$ and $sv(t)$.

4.4.3.3 Complex Depth-Control Structures

The equations describing the flow through complex depth-control structures may be constructed by simply adding the equations corresponding to each outlet of the structure. For example, if a depth-control structure has three segments; two identical Type 1(a)'s and one Type 2(a), the equation describing the flow through the structure becomes:

$$\begin{aligned} Q(t) = {} & \lfloor Ra(pcs(osh_u(t_0), fc(t_0), cp(t_0)), ufc(t))]_1 \\ & + \lfloor Ra(pcs(osh_u(t_0), fc(t_0), cp(t_0)), ufc(t))\rfloor_2 \\ & + [Ra(pcs(osh_u(t_0, t), fc(t_0, t), cp(t_0, t)), ufc(t))]_3 \end{aligned} \tag{4.59}$$

The first two terms pertain to the Type 1(a) outlets and the last term pertains to the Type 2(a) outlet.

4.4.4 Summary of Depth-Control Structure Types

The classification of depth-control structures is summarized in Table 4.3. An asterisk beside the type number indicates that examples of this type of structure do not exist.

4.5 Storage Facilities

4.5.1 Water Management Function

Water storage facilities are used in irrigation conveyance systems to serve a variety of functions. These include water supply management through the use of large internal reservoirs, water flow rate management through the use of balancing (regulating) ponds, and the expedition of diversion operations employing pump-motor assemblies through the use of sump-storage. A brief introduction to the terminology and theory used to describe individual water storage facilities is given in Appendix D.

4.5.2 Internal Reservoirs

Internal reservoirs receive and store large volumes of water during periods of supply surplus for use during periods of supply shortfall. It is important to note that the water supply to an internal reservoir is limited not only by the availability of water at the primary water supply but also by the capacity of the conveyance system to transport water from the location of primary water supply to the internal

Table 4.3 Depth-Control Structure Classification System

Type No.	Hydraulic Control	Method of Operation	Type of Control
1	upstream	manual/remote	open-loop
2	upstream	automatic	open-loop
3	upstream	fixed	open-loop
4	upstream	fixed-automatic	open-loop
5	upstream/downstream	manual/remote	open-loop
6	upstream/downstream	automatic	open-loop
7	upstream/downstream	fixed	open-loop
8	upstream/downstream	fixed-automatic	open-loop
9	positive	manual/remote	open-loop
10	positive	automatic	open-loop
11	positive	fixed	open-loop
12	positive	fixed-automatic	open-loop
13	upstream	manual/remote	closed-loop
14	upstream	automatic	closed-loop
15 *	upstream	fixed	closed-loop
16	upstream	fixed-automatic	closed-loop
17	upstream/downstream	manual/remote	closed-loop
18	upstream/downstream	automatic	closed-loop
19 *	upstream/downstream	fixed	closed-loop
20	upstream/downstream	fixed-automatic	closed-loop
21	positive	manual/remote	closed-loop
22	positive	automatic	closed-loop
23 *	positive	fixed	closed-loop
24	positive	fixed-automatic	closed-loop

reservoir (see Figure 4.10). During periods of maximum irrigation water consumption, diversion from an internal reservoir may greatly exceed the rate of water delivery to it and the volume of water stored in the reservoir will be depleted.

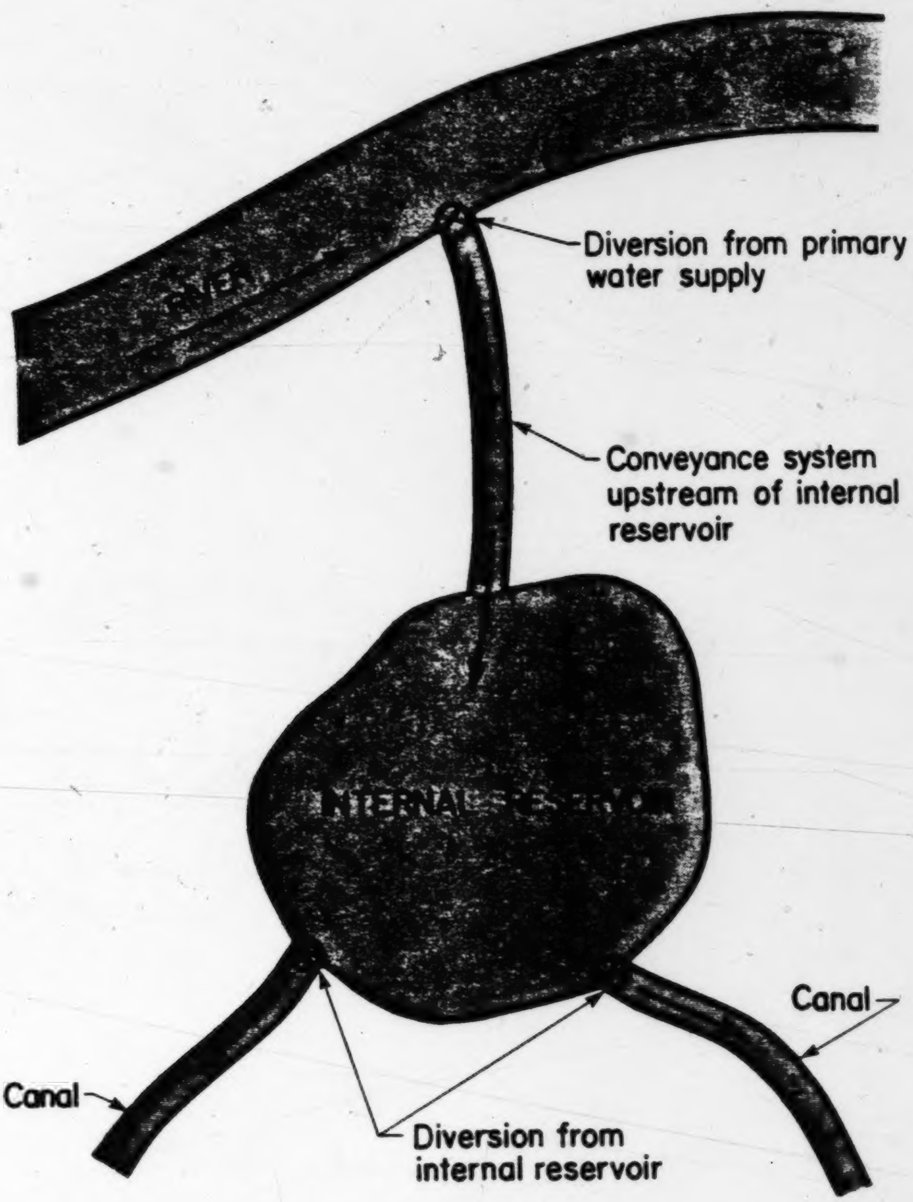
4.5.3 Balancing Ponds/(Regulating Reservoirs)

Balancing ponds are intended to suppress the detrimental impacts of fluctuations in water flow rate through difficult-to-control diversion structures. A balancing pond consists of a small storage reservoir located downstream of the problem diversion structure as illustrated in Figure 4.11. Downstream water demands are met by other diversion structures located on the balancing pond itself. These diversion structures normally employ upstream hydraulic control although other types of hydraulic control may be used. Rate of inflow to the balancing pond may fluctuate widely without significantly affecting the depth of water in the balancing pond so that control over the water diversions from the balancing pond can be maintained.

Balancing ponds are not intended to provide significant volumes of water storage and the volume of water delivery to balancing ponds must coincide more closely in time with the volume of water delivered from them. Use of balancing ponds may remove the need to use automatically controlled diversion structures.

4.5.4 Sump-Storage

Sump-storage is used to facilitate the operation of pump-motor assemblies. Sump-storage may not only provide the necessary submergence



RIVER
Diversion from primary water supply
Conveyance system upstream of internal reservoir
INTERNAL RESERVOIR
Canal
Canal
Diversion from internal reservoir

Figure 4.10 Schematic of typical internal reservoir.

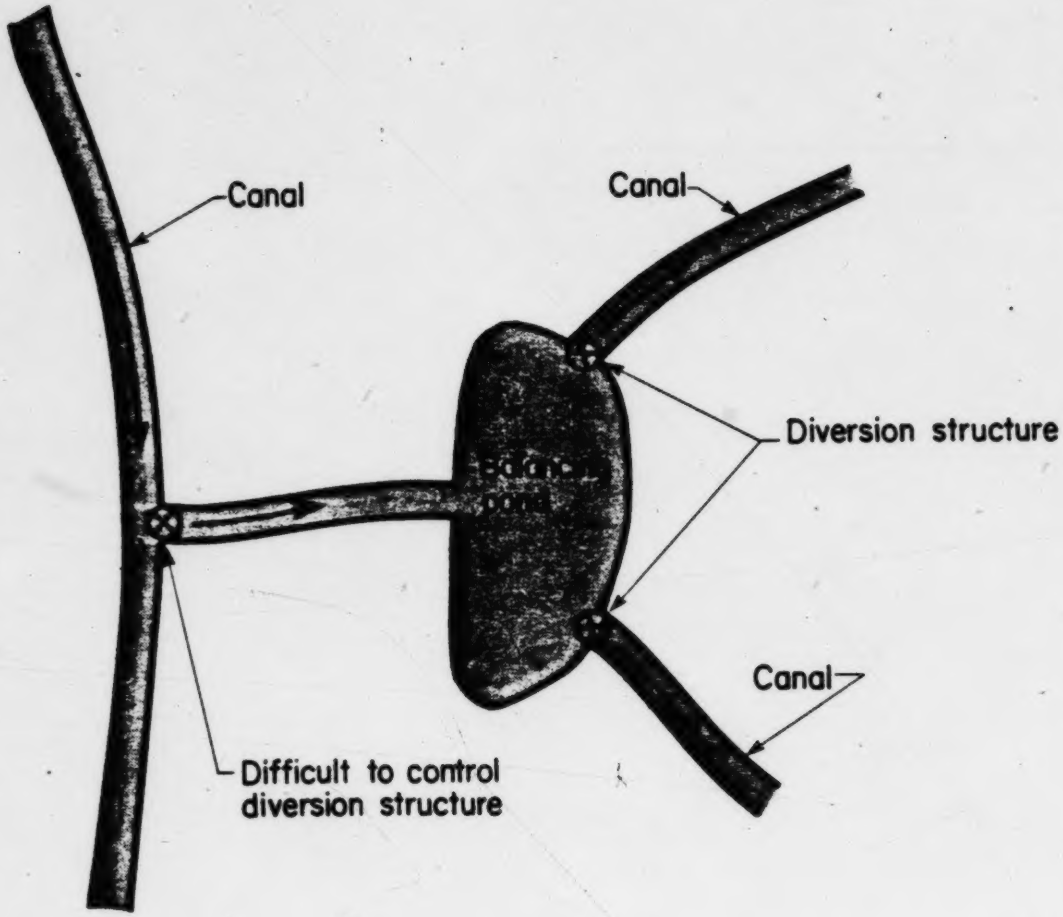

Canal
Canal
Diversion structure
Canal
Difficult to control
diversion structure

Figure 4.11 Schematic of typical balancing pond (or regulating reservoir).

of the pump intake but may provide a balancing pond function as well.

4.5.5 Level-Pond Operation of Storage Facilities

Rate of water flow into a water storage facility may be controlled by the depth of water in the storage facility itself. This method of inflow control is called level-pond operation and is frequently used in conjunction with sump-storage and small internal reservoirs which are part of quaternary or distributary subsystems. A typical installation is illustrated in Figure 4.12. The relationship between the maximum allowable level of water in the storage facility and the maximum possible level of water in the canal supplying the water is such that there is no possible danger of the storage facility overflowing. The diversion structure linking the supply canal to the storage facility obviously employs upstream/downstream hydraulic control.

4.5.6 Discussion

The operation and use of the various types of storage facilities has been described. However it should be noted that both internal reservoirs and balancing ponds will provide adequate sump-storage, and internal reservoirs will also provide the balancing pond function.

4.6 Compound Components

4.6.1 Description

Compound conveyance system components combine two or more water management functions into a single continuous structure a practice which

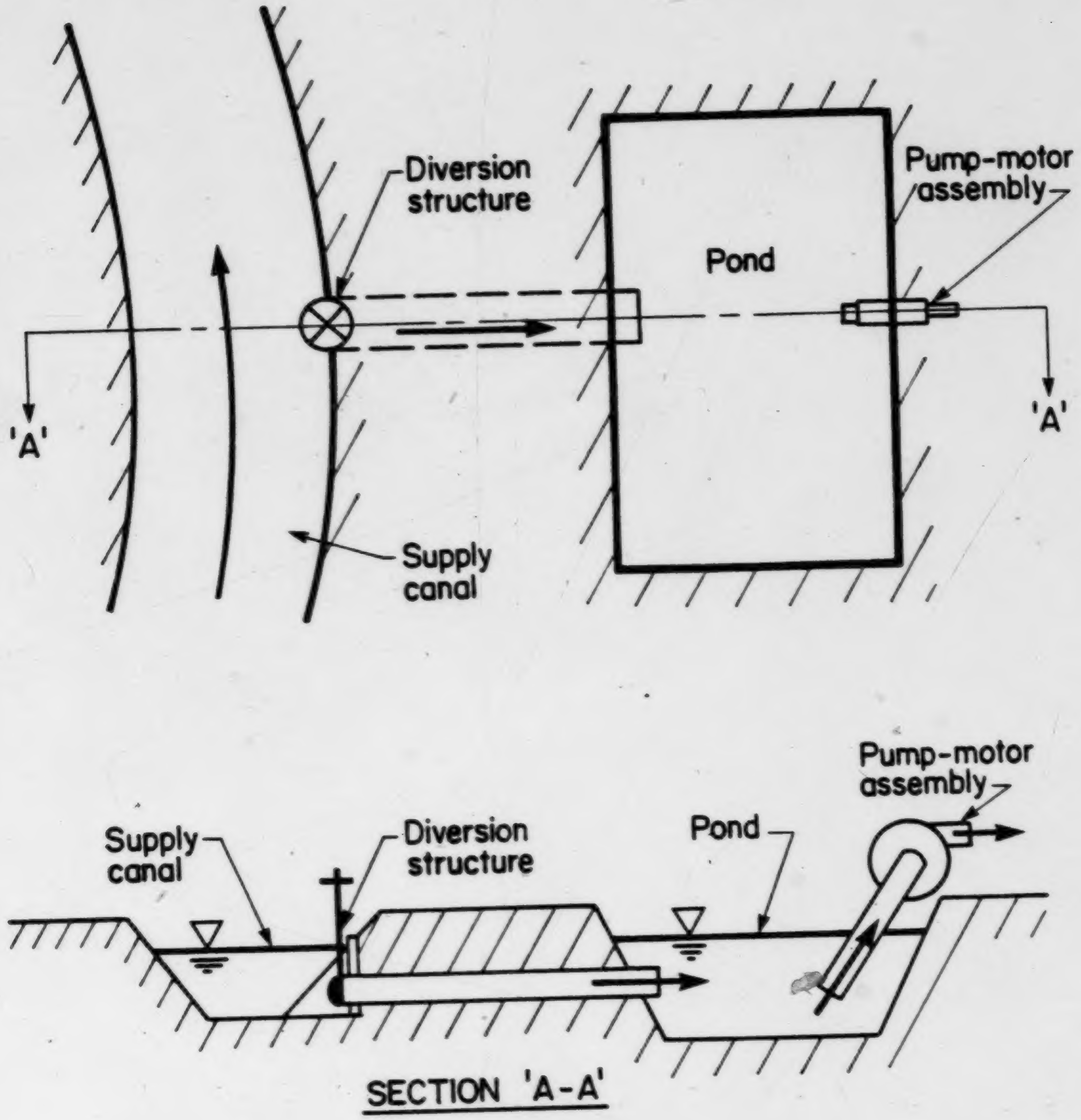

Diversion structure
Pump-motor assembly
Pond
'A'
'A'
Supply canal
Pump-motor assembly
Supply canal
Diversion structure
Pond
SECTION 'A-A'

Figure 4.12 Schematic showing level-pond operation of storage facility.

frequently results in considerable cost savings due to the shared use of structural elements. Compound components normally combine diversion or depth-control structures with canal- or pipeline-associated specialty structures but many other combinations are possible and used. Analysis of compound components presents few difficulties as the individual components comprising the compound component are easily distinguished and described using the classification system presented earlier in this chapter.

4.6.2 Examples

Two very common examples of compound components involve combinations of depth-control and water transfer and diversion and water transfer. Examples of both combinations are presented.

4.6.2.1 Depth-Control and Water Transfer Combination

An example of a compound component which combines the depth-control and transfer function is illustrated in Figure 4.13. This structure combines a Type 1(a) depth-control structure with a canal-associated specialty structure to accommodate a negative change in canal bed elevation. Numerous variations of this structure, commonly called vertical check-drops, are used especially in association with concrete-lined canals where rapid energy dissipation may not be as important.

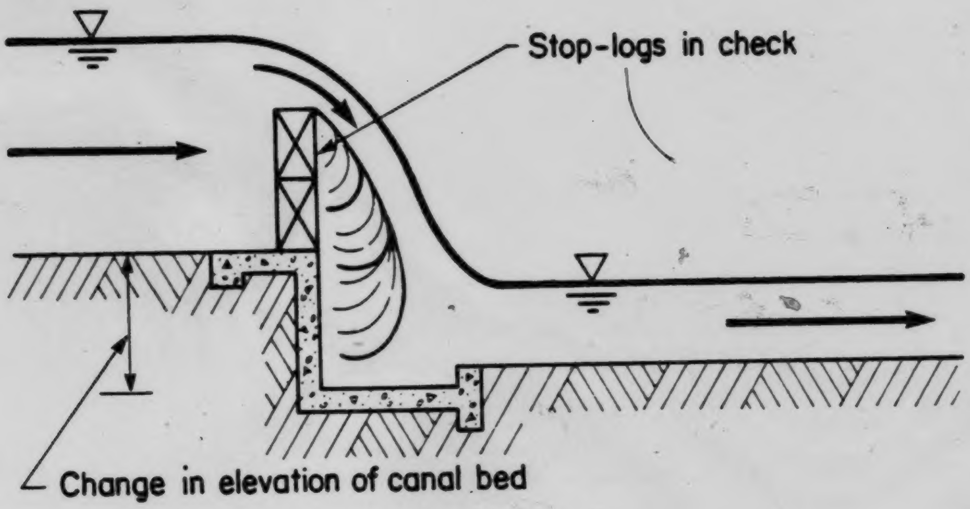

Stop-logs in check
Change in elevation of canal bed

Figure 4.13 Example of compound component which combines a Type 1(a) depth-control structure with a drop structure.

4.6.2.2 Diversion-Water Transfer Combination

An example of a compound component which combines diversion and transfer functions is illustrated in Figure 4.14. The radial gate, which performs the diversion function, is a Type 1 diversion structure. The chute-stilling basin portion is really two canal associated specialty structures operating in series. The chute safely lowers the water from the higher elevation canal bed into the stilling basin which dissipates the kinetic energy gained in the lowering process. Numerous variations of this structure, commonly called gated drop-structures are used.

4.7 Multifunction Components

Multifunction components perform two or more water management functions using a single conveyance system component. Multifunction components differ from compound components in that structural additions are not required for each function to be performed. That is, each function requires and uses the same structural elements.

Examples of commonly used multifunction components include:

1. diversion and depth-control, (e.g. drains from canals and open pipeline systems);
2. diversion and water transfer, (e.g. drains from canals and open pipeline systems);
3. depth-control and water transfer, (e.g. lift-stations);
4. storage and water transfer, (e.g. level-top canals); and,

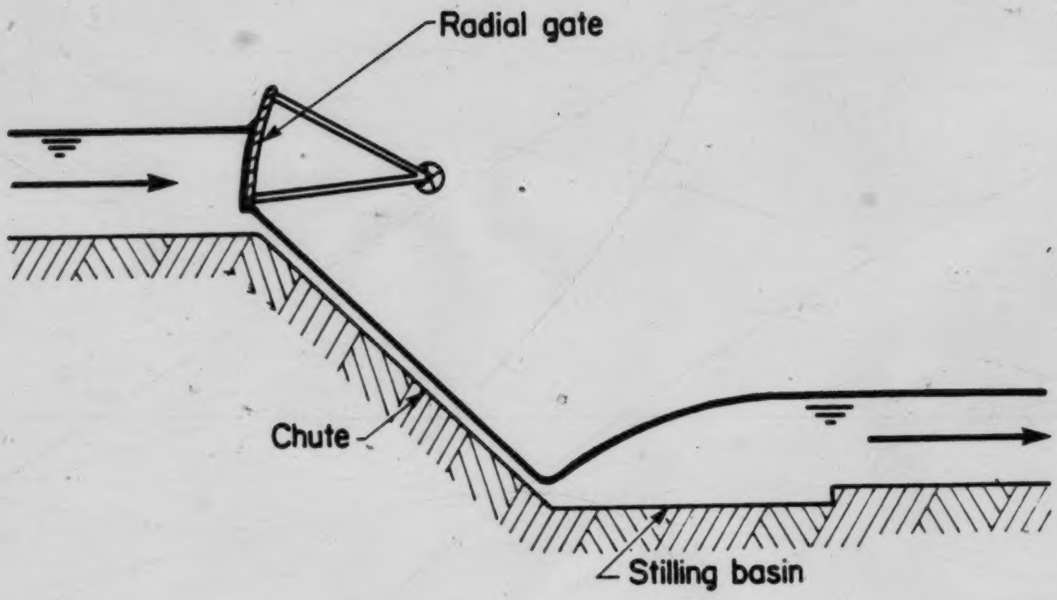

Radial gate
Chute
Stilling basin

Figure 4.14 Example of a compound component which combines a Type 1 diversion structure and a chute-stilling basin.

5. water measurement and diversion, depth control or other water transfer functions, (e.g. nearly any water diversion, depth-control or transfer component).

4.8 Discussion

The proposed system for classification of irrigation conveyance system components succeeds in providing a method with which to identify, describe and compare the numerous and diverse components found within irrigation conveyance systems. Use of the proposed classification system offers the following advantages:

1. The classification system is comprehensive. All structural components found within irrigation conveyance systems may be considered.
2. Use of the classification system facilitates the concise description of component and conveyance system operations.
3. The classification system is readily expanded to enable the consideration of as much of detail individual investigations may require.
4. The classification system not only provides the framework but also suggests the criteria with which to examine and evaluate individual conveyance system components.
5. Use of the classification system facilitates the conceptualization and development of mathematical models of conveyance systems and subsystems by providing a guide by which investigators can recognize individual component

function and identify their significant hydraulic and operational characteristics.

CHAPTER 5

TERMINOLOGY FOR DESCRIBING IRRIGATION CONVEYANCE SYSTEMS

5.1 Introduction

A terminology useful for describing irrigation conveyance systems is presented. It is modelled after the one introduced by the International Commission on Irrigation and Drainage (1978). However, significant modifications were made to this work for greater comprehensiveness and to remove ambiguity. These improvements were made possible largely because of the introduction of the system for classifying irrigation conveyance system components presented in Chapter 4.

5.2 General Description of Terminology

The terminology presented is based on a classification of irrigation conveyance subsystems which are identified by their relationship to the irrigation water demands they are intended to satisfy. Three general types of subsystems are identified: quaternary, distributary and lateral. The lateral classification may be expanded as required as is described later in this chapter. Water demand is determined by consideration of the nature of the land area being irrigated and by the consideration of the number and types of diversion structures being supplied.

For each subsystem type, only those conveyance system components which describe diversion, long distance transfer and storage of the water are specifically identified. It may be noted that if the flow conditions within the components which are identified are known, the operation of the conveyance systems is fully described.

No consideration is given to the management/ownership characteristics of conveyance systems or relationships to water supply.

5.3 Quaternary Subsystems

Quaternary conveyance subsystems facilitate the delivery of water to single land areas known as quaternary units. Quaternary units are defined as the area of land that can be irrigated efficiently by one man if he were to receive the flow continuously. All portions of the land area constituting the quaternary unit are contiguous and the entire unit is normally irrigated in one application cycle. Water is delivered to a quaternary unit through a single diversion structure known as the primary quaternary off-take, P.Q.OT. Conveyance of water on quaternary units is accomplished by the quaternary canals/pipelines, Q.C/P. Water may be temporarily stored in a quaternary storage facility, Q.St., for later irrigation use on the quaternary unit. Water may enter the quaternary storage facility either directly or through a primary quaternary off-take, or a quaternary storage inflow structure, Q.SI. Water may be removed from the quaternary storage facility through a single diversion structure known as a secondary quaternary off-take, S.Q.OT., either for immediate application to the land or for further distribution on the quaternary unit. Secondary quaternary off-takes may also be employed along a quaternary canal/pipeline immediately upstream of a quaternary spill structure. Water leaving the secondary quaternary off-take, in this instance, may be further distributed on the quaternary unit. Water delivered to the quaternary unit and not applied to the land may be discharged, through a diversion structure called a quaternary spill structure, Q.Sp, into a drainage system where it is no longer available for application to the land, or to another portion of the conveyance system. Diversion structures, which permit water from surface drainage, spill from other portions of the conveyance system or drainage from any other source, to enter the quaternary canal/pipeline or storage facility are called quaternary drainage inflow structures, Q.DI.

Examples of quaternary subsystems are shown in Figures 5.1(a) to (g). Figure 5.1(a) illustrates a quaternary subsystem typical of quaternary units where surface irrigation application techniques are employed. Figure 5.1(b) illustrates a quaternary subsystem where water is either pumped or received under pressure and enters a sprinkler application system directly, or where the entire quaternary unit is flooded from a single quaternary off-take as in basin irrigation. Figure 5.1(c) illustrates a quaternary unit which employs a storage facility. In this example water is delivered to the quaternary storage facility by canal or pipeline. Water is removed from the quaternary storage facility for application to the land through a secondary quaternary off-take such as a pump. Excess water is spilled from the storage facility through a quaternary spill structure. Figure 5.1(d)

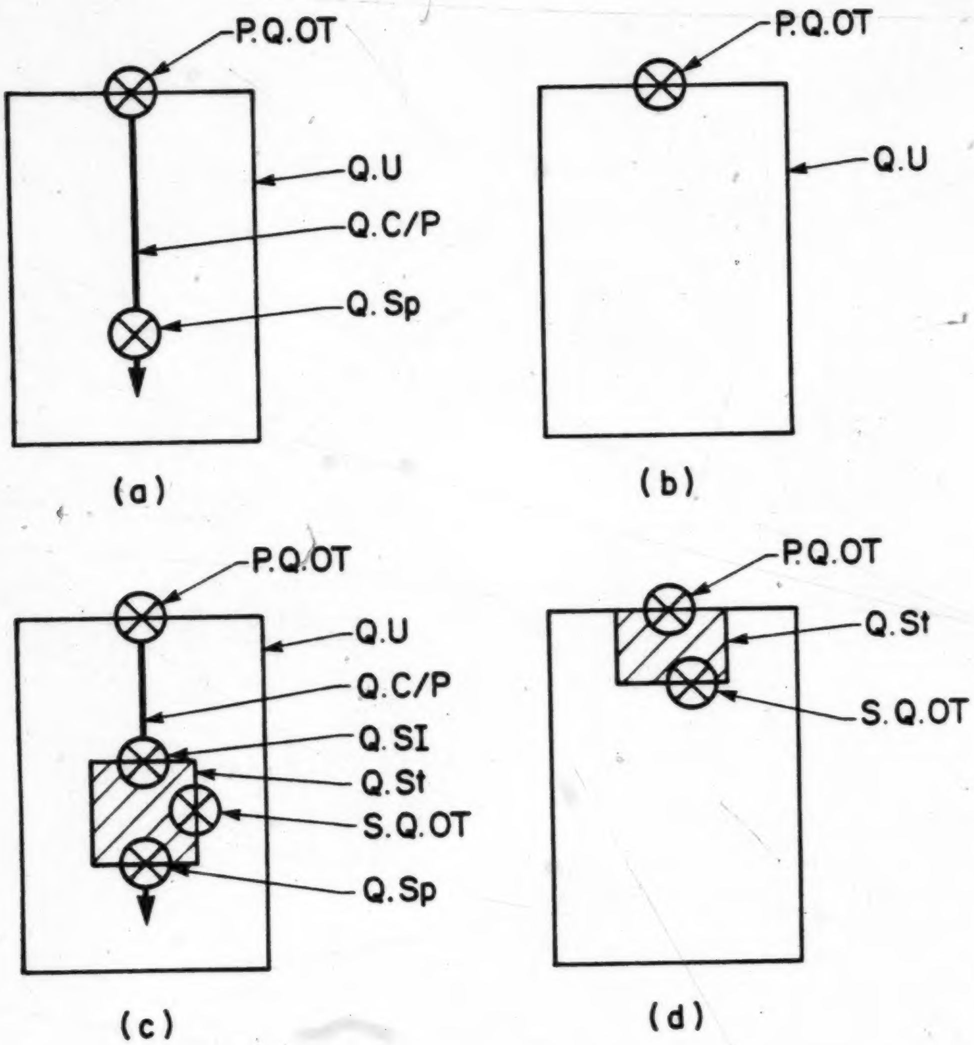

P.Q.OT
Q.U
Q.C/P
Q.Sp
(a)
P.Q.OT
Q.U
(b)
P.Q.OT
Q.U
Q.C/P
Q.SI
Q.St
S.Q.OT
Q.Sp
(c)
P.Q.OT
Q.St
S.Q.OT
(d)

Figure 5.1 (a) - (d) Examples of quaternary subsystems.

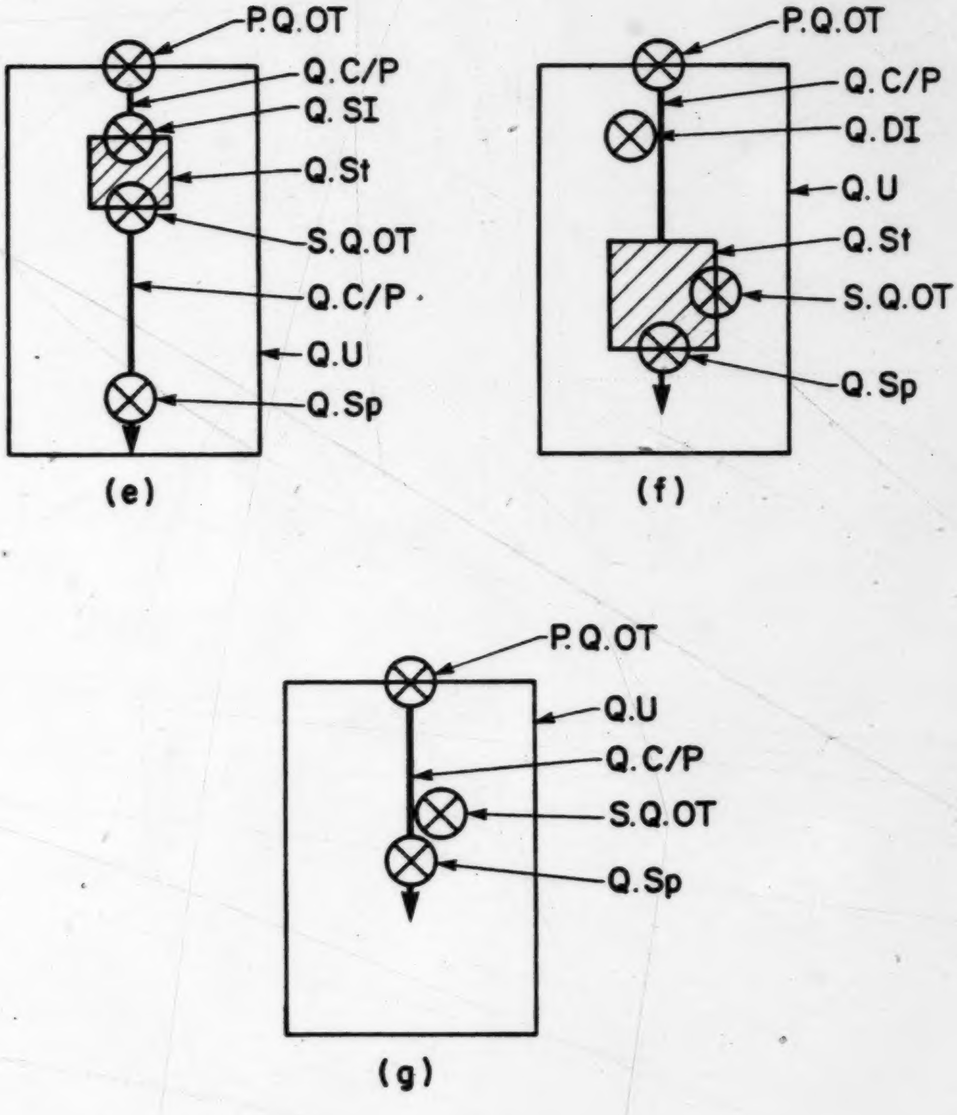

P.Q.OT
Q.C/P
Q.SI
Q.St
S.Q.OT
Q.C/P
Q.U
Q.Sp
(e)
P.Q.OT
Q.C/P
Q.DI
Q.U
Q.St
S.Q.OT
Q.Sp
(f)
P.Q.OT
Q.U
Q.C/P
S.Q.OT
Q.Sp
(g)

Figure 5.1 (e) - (g) Examples of quaternary subsystems.

illustrates a quaternary unit which employs a storage facility using level-pond operation. The secondary quaternary off-take is likely a pump. Figure 5.1(e) illustrates a quaternary unit which employs two reaches of canal/pipeline. A surface-type application system is likely used. Figure 5.1(f) illustrates a quaternary unit whose canal/pipeline is receiving additional water from a quaternary drainage inflow structure. Note that water enters the quaternary storage facility directly. Figure 5.1(g) illustrates the relationship between a secondary quaternary off-take and a quaternary spill structure when no storage facility is used.

5.4 Distributary Subsystems

Distributary conveyance subsystems facilitate the delivery of water to the primary off-takes of two or more quaternary units whose combined land area is named the distributary unit, D.U. Water is delivered to a single distributary unit through a single diversion structure called the primary distributary off-take, P.D.OT. Distributary canals/pipelines, D.C/P, transport the water from the primary distributary off-take to primary quaternary off-takes, a distributary storage facility, D.St., or a secondary distributary off-take, S.D.OT. Water may enter the distributary storage facility through a distributary storage inflow structure, D.S.I. and be withdrawn through any number of primary quaternary off-takes and/or through a single secondary distributary off-take for further distribution. Water delivered to the distributary unit and not distributed to primary quaternary off-takes or stored in the distributary storage facility may be discharged through a diversion structure called a distributary spill structure, D.Sp., into a drainage system where it is no longer available for application to the land, or to another portion of the conveyance system. Diversion structures, which permit water from surface drainage, spill from other portions of the conveyance system or drainage from any other source, to enter the distributary canal/pipeline or storage facility are called distributary drainage inflow structures, D.DI.

Examples of distributary subsystems are shown in Figures 5.2(a) to (h). Figure 5.2(a) illustrates a distributary subsystem which includes only two primary quaternary off-takes and their associated quaternary units indicated by the dashed lines. This system includes a distributary spill structure which indicates that it is likely a canal or open-pipeline system. Distributary subsystems employing closed pipeline systems would not usually include a spill structure. Figure 5.2(b) illustrates a distributary unit composed of four quaternary units. Note the storage facility and storage inflow structure. Figure 5.2(c) illustrates a distributary subsystem which incorporates a storage facility using in level pond operational mode. Figure 5.2(a) illustrates a distributary subsystem which includes a drainage inflow structure. Note that water spills directly into the storage facility without requiring a special storage inflow structure. Figure 5.2(e) illustrates a distributary subsystem typical of those which are used with a closed pipeline system pressurized with water pumped from a level pond. Figure 5.2(f) illustrates the relationship between a secondary distributary off-take and a primary distributary off-take when no

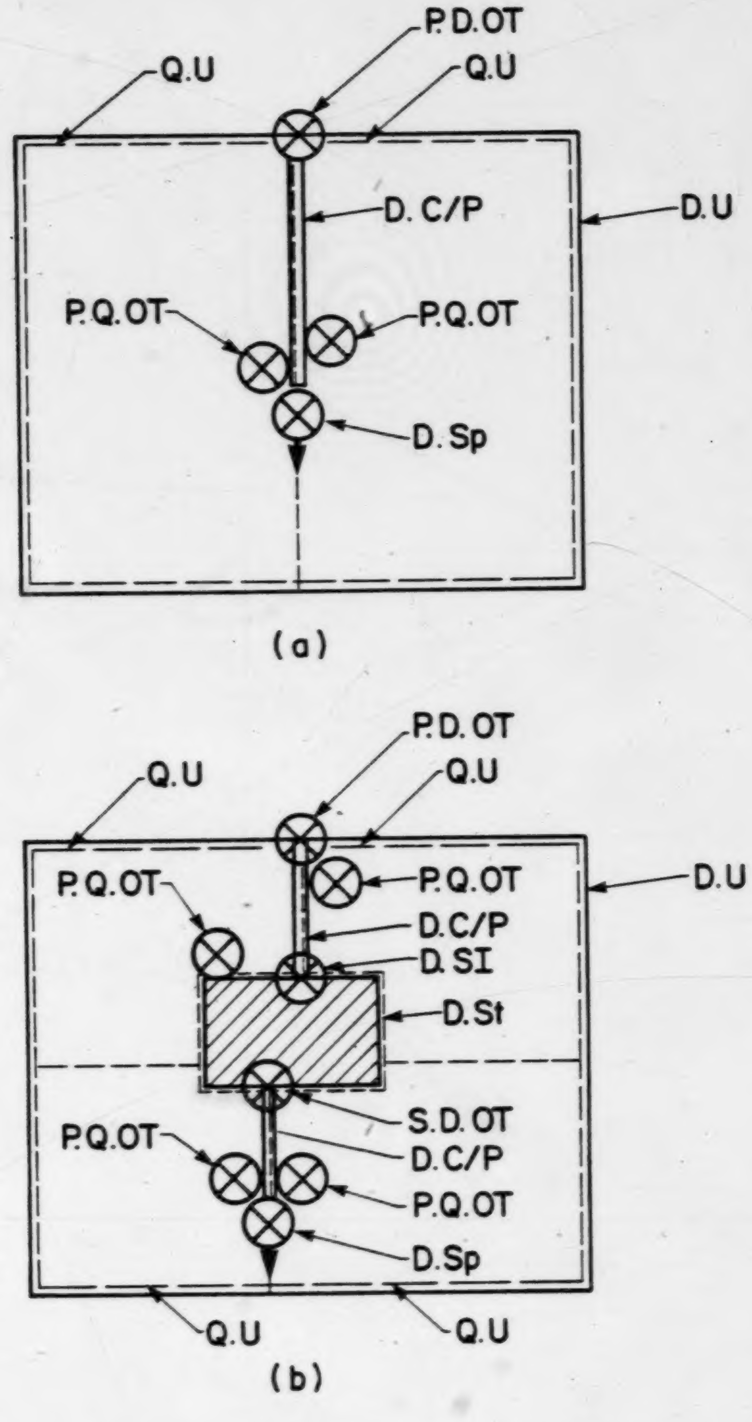

P.D.OT
Q.U
Q.U
D. C/P
D.U
P.Q.OT
P.Q.OT
D. Sp
(a)
P.D.OT
Q.U
Q.U
P.Q.OT
P.Q.OT
D.U
D.C/P
D.SI
D.St
S.D.OT
P.Q.OT
D.C/P
P.Q.OT
D.Sp
Q.U
Q.U
(b)

Figure 5.2 (a) and (b) Examples of distributary subsystems.

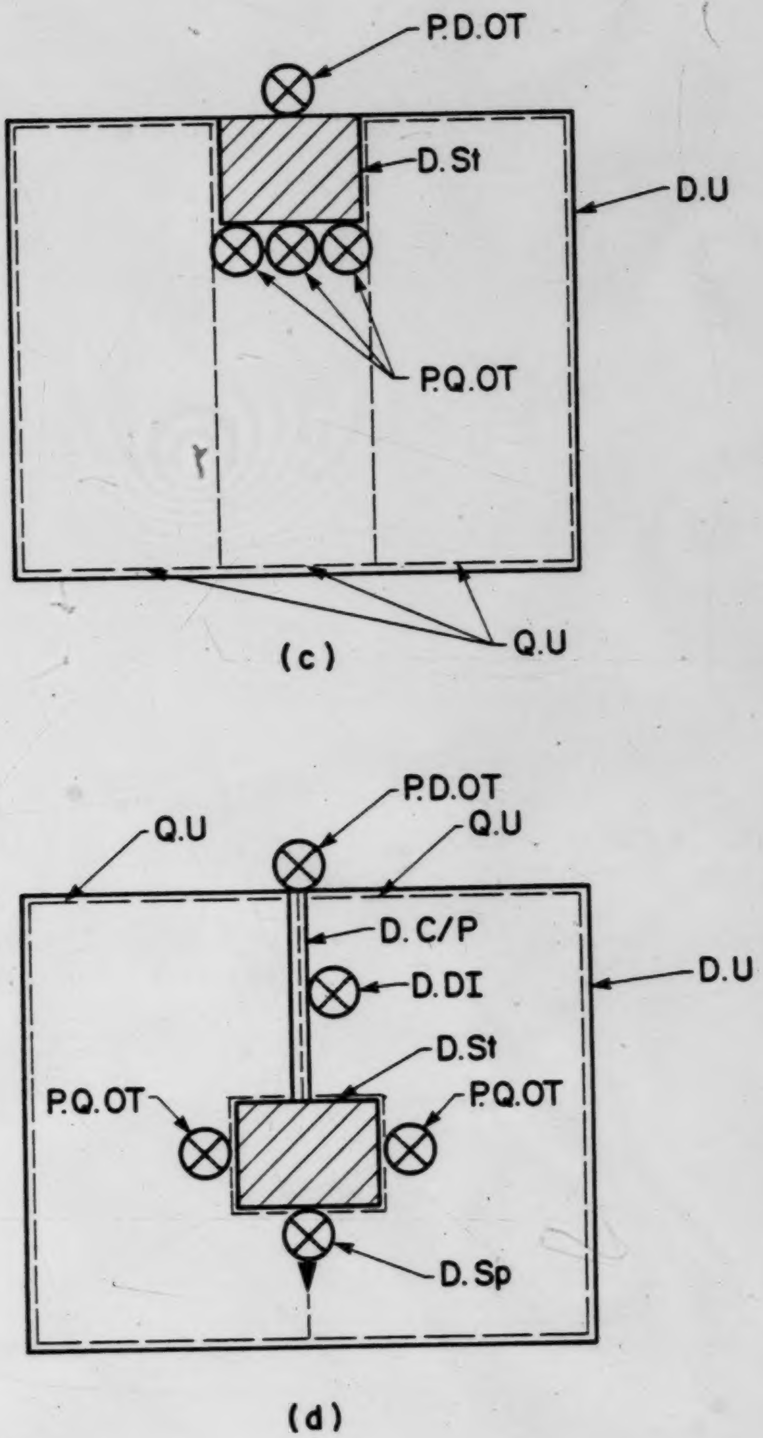
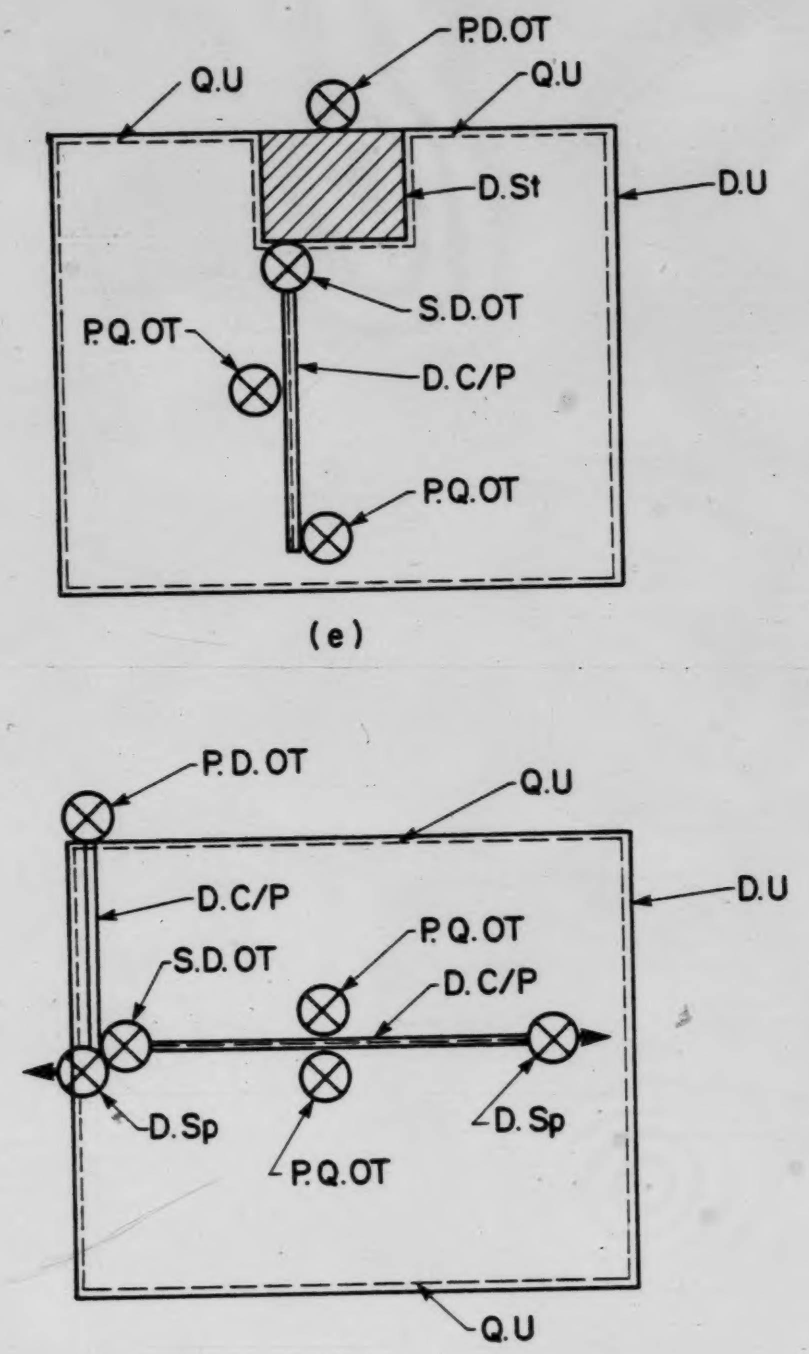

P.D.OT
D.St
D.U
P.Q.OT
Q.U
(c)
P.D.OT
Q.U
Q.U
D.C/P
D.DI
D.U
D.St
P.Q.OT
P.Q.OT
D.Sp
(d)

Figure 5.2 (c) and (d) Examples of distributary subsystems.


P.D.OT
Q.U
Q.U
D.St
D.U
S.D.OT
P.Q.OT
D.C/P
P.Q.OT
P.D.OT
Q.U
D.C/P
D.U
P.Q.OT
S.D.OT
D.C/P
D.Sp
D.Sp
P.Q.OT
Q.U

(e)

(f)

Figure 5.2 (e) and (f) Examples of distributary subsystems.

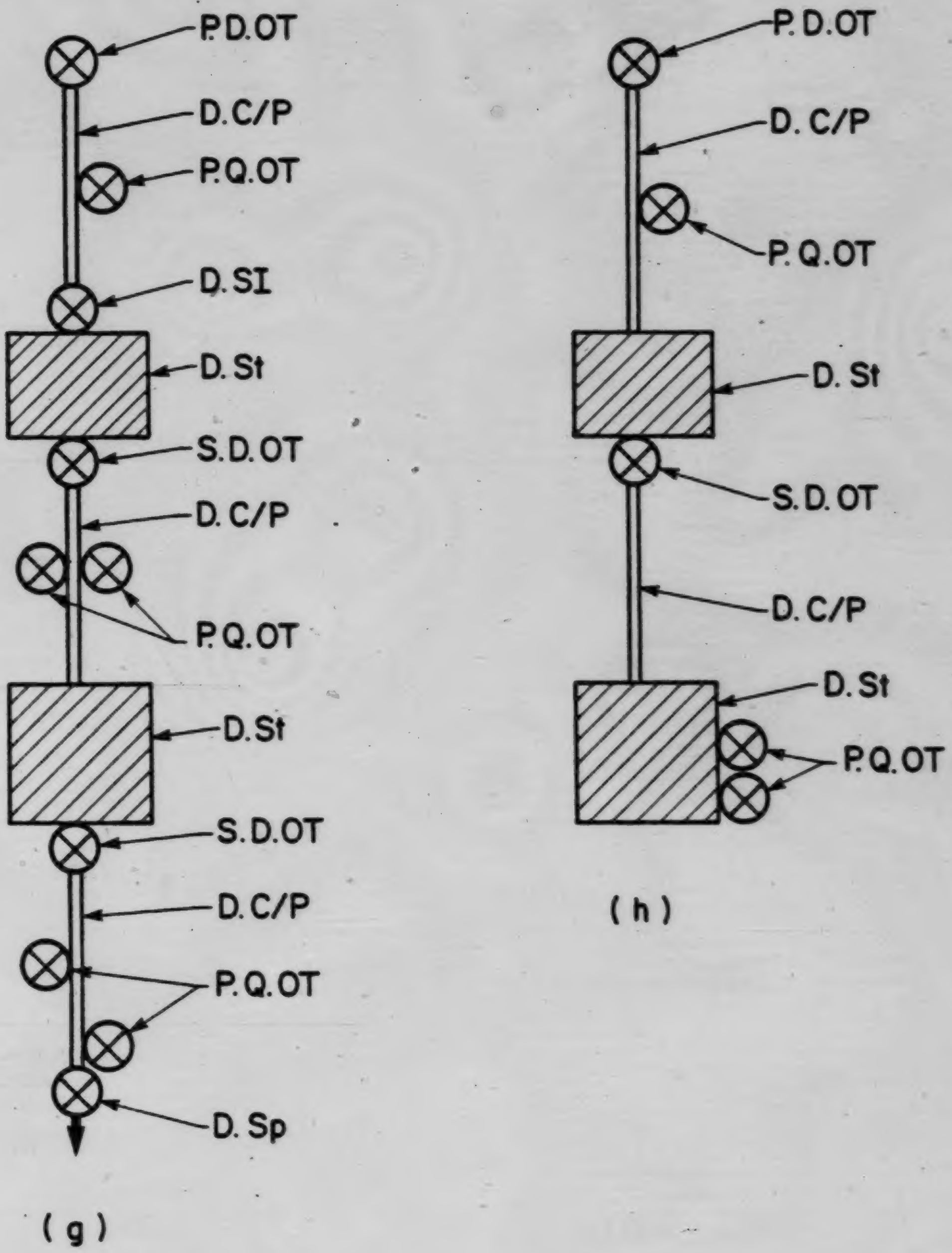

P.D.OT
D.C/P
P.Q.OT
D.SI
D.St
S.D.OT
D.C/P
P.Q.OT
D.St
S.D.OT
D.C/P
P.Q.OT
D.Sp
(g)
P.D.OT
D.C/P
P.Q.OT
D.St
S.D.OT
D.C/P
D.St
P.Q.OT
(h)

Figure 5.2 (g) and (h) Examples of distributary subsystems.

storage facility is used. Figure 5.2(g) illustrates a distributary subsystem which includes two storage facilities. Note that only one storage facility employs a storage inflow structure. Figure 5.2(h) illustrates a distributary subsystem which includes two storage facilities and no spill structure. Many other examples of distributary subsystems of varying complexity may be derived.

5.5 First-Order Lateral Subsystems

First order lateral conveyance subsystems, L1, facilitate the delivery of water to the primary off-takes of two or more distributary units plus the primary off-takes of any number of quaternary units, or to the primary off-takes of only one distributary unit and one or more quaternary units. The combined area being supplied with water is known as the L1 unit, L1.U. Water is delivered to an L1 unit through a single diversion structure called the primary L1 off-take, P.L1.OT. L1 canals/pipelines, L1.C/P., transport the water from the primary L1 off-take to the primary distributary and quaternary off-takes or to the L1 storage facility, L1.St. Water may enter the L1 storage facility through an L1 storage inflow structure, L1.SI., and be withdrawn through any number of primary quaternary and distributary off-takes and/or through a single secondary L1 off-take for further distribution. Water delivered to an L1 unit and not used may be discharged through a diversion structure called an L1 spill, L1.Sp., into a drainage system where it is no longer available for application to the land; or, to another portion of the conveyance system. Diversion structures which permit water from surface drainage, spill from other portions of the conveyance system or drainage from any other source, to enter the distributary canal/pipeline or storage facility are called L1 drainage inflow structures, L1.DI.

Examples of L1 conveyance subsystems are shown in Figures 5.3(a) to (o). Figures 5.3(a) and (b) illustrate minimum L1 subsystems typical of those which employ closed pipeline systems. Figures 5.3(c) and (d) illustrate the same subsystems with additional quaternary off-takes but with the minimum number of distributary off-takes. Figures 5.3(e) and (f) illustrate minimum L1 subsystems typical of those which employ canals. Note the drainage inflow and spill structures. Figure 5.3(g) illustrates an L1 subsystem which delivers water to two distributary off-takes which may be part of a compound diversion structure known as a division structure or flow splitter. The division may be more than two ways if required. Note that there is no spill structure. Figure 5.3(h) illustrates an L1 subsystem which delivers water to two distributary off-takes. Because of the inclusion of the spill structure, the two distributary off-takes do not constitute a division point. Figure 5.3(i) simply illustrates a more typical L1 subsystem. Figure 5.3(j) illustrates the application of a secondary L1 off-take without use of an L1 storage facility. Figure 5.3(k) illustrates an example of the typical use of an L1 storage facility. Note the use of an L1 storage inflow structure. Figure 5.3(l) illustrates one of the smallest L1 subsystems which employs a canal/pipeline and a storage facility. Note that the previous two examples employed spill structures. Figures 5.3(m) and (n) illustrate the use of L1 storage facilities employed as

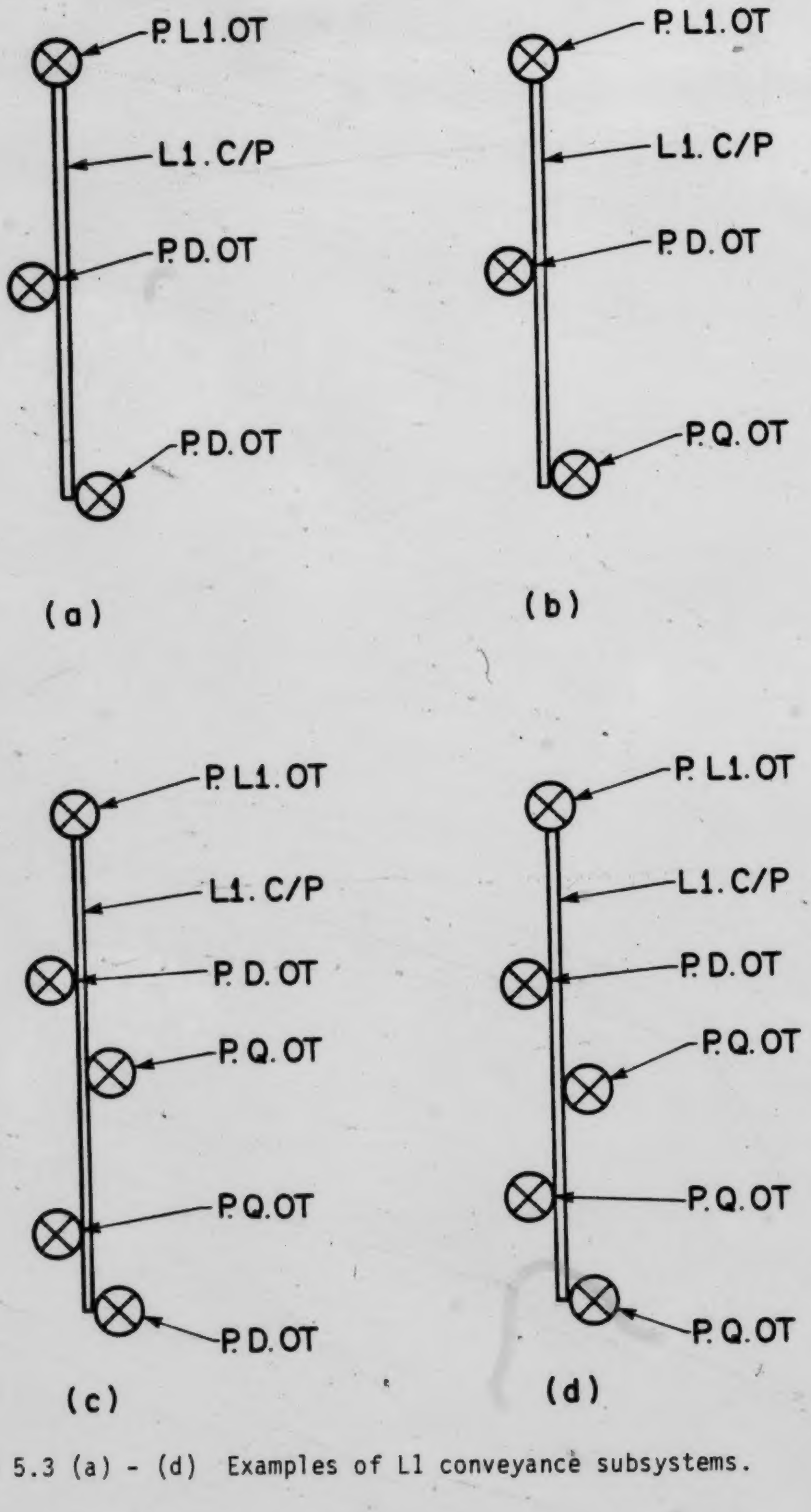

P. L1.OT
L1.C/P
P. D.OT
P. D.OT
(a)
P. L1.OT
L1. C/P
P. D.OT
P. Q.OT
(b)
P. L1.OT
L1. C/P
P. D.OT
P. Q.OT
P. Q.OT
P. D.OT
(c)
P. L1.OT
L1.C/P
P. D.OT
P. Q.OT
P. Q.OT
P. Q.OT
(d)

Figure 5.3 (a) - (d) Examples of L1 conveyance subsystems.

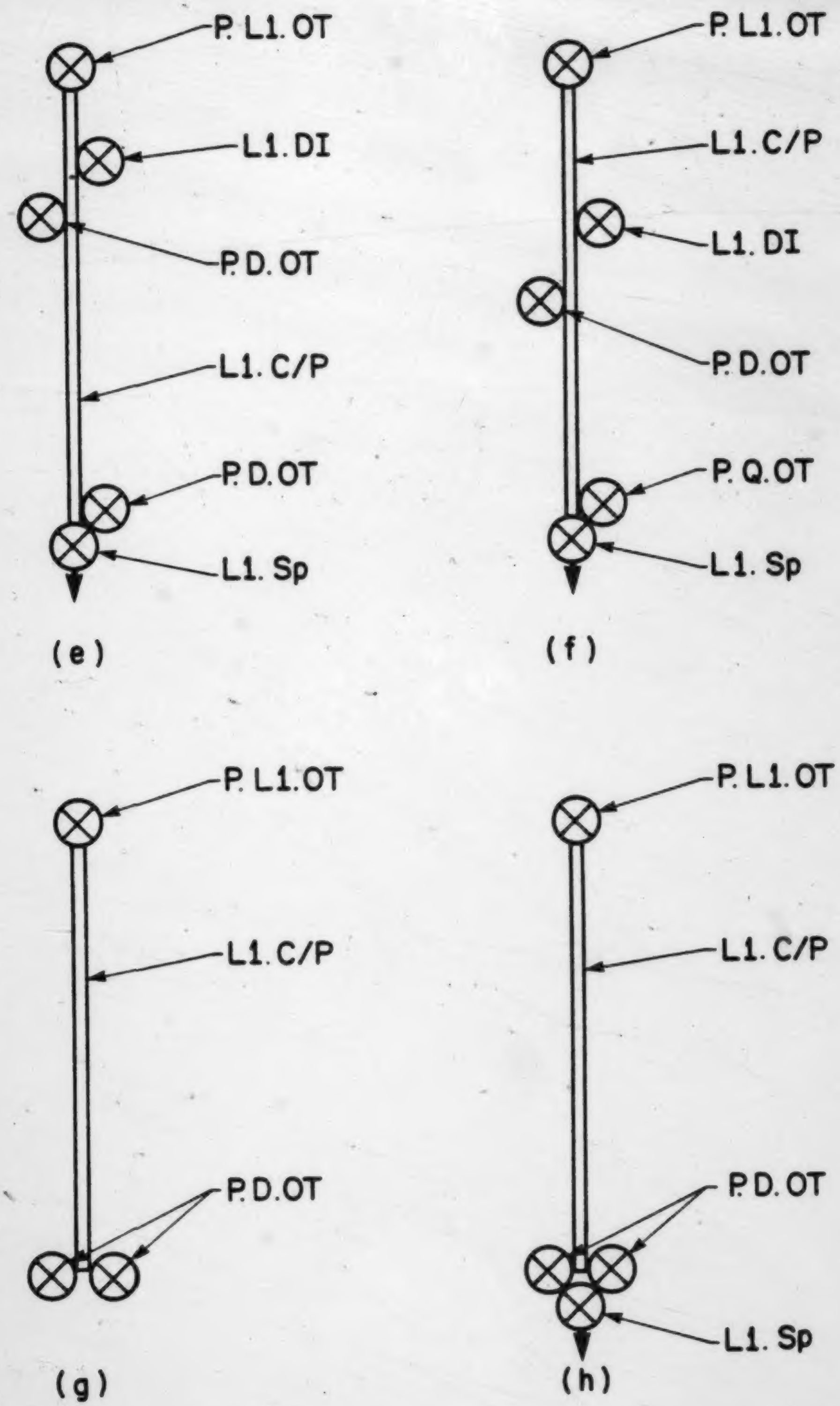

P. L1. OT
L1. DI
P. D. OT
L1. C/P
P. D. OT
L1. Sp
(e)
P. L1. OT
L1. C/P
L1. DI
P. D. OT
P. Q. OT
L1. Sp
(f)
P. L1. OT
L1. C/P
P. D. OT
(g)
P. L1. OT
L1. C/P
P. D. OT
L1. Sp
(h)

Figure 5.3 (e) - (h) Examples of L1 conveyance subsystems.

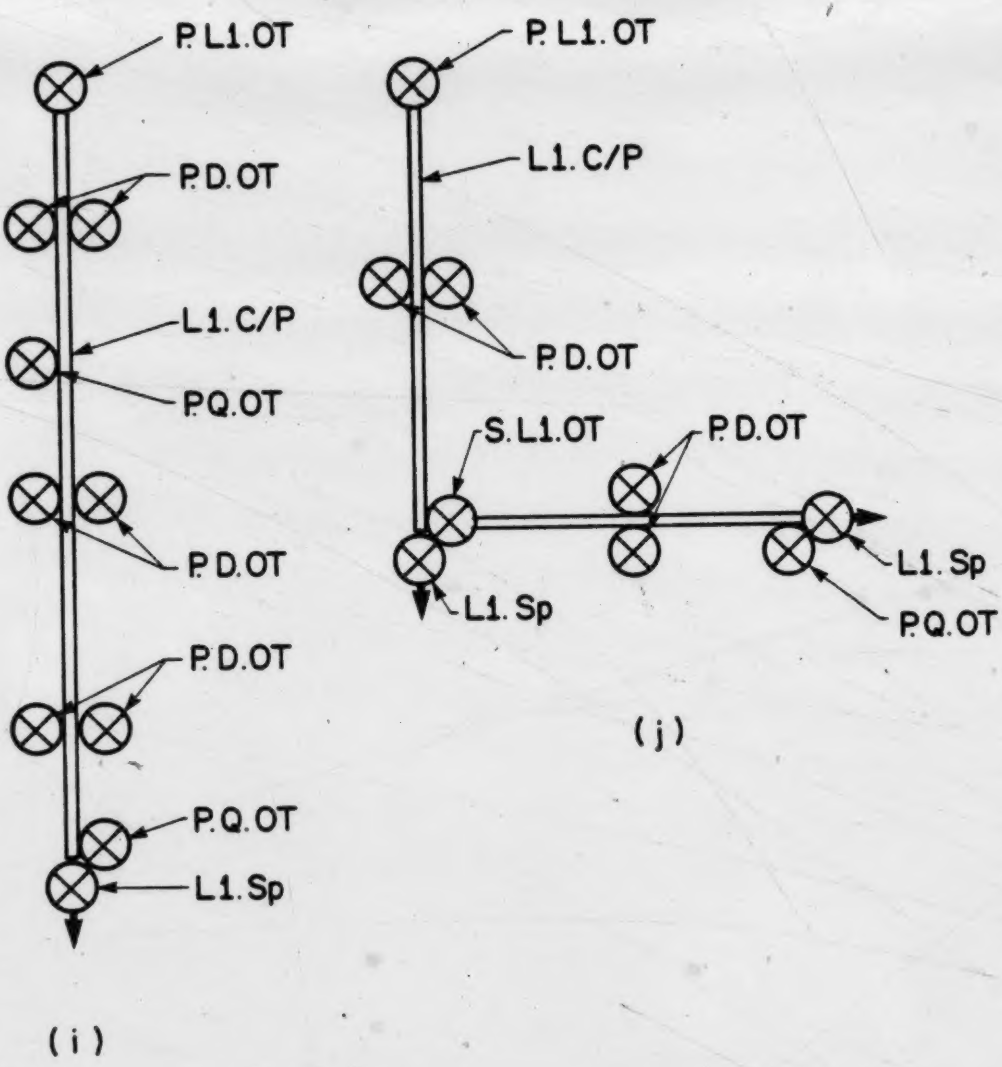

P. L1.OT
P. D.OT
L1. C/P
P.Q.OT
P. D.OT
P. D.OT
P.Q.OT
L1.Sp
(i)
P. L1. OT
L1. C/P
P. D.OT
S. L1.OT
P. D.OT
L1. Sp
L1. Sp
P.Q.OT
(j)

Figure 5.3 (i) and (j) Examples of L1 conveyance subsystems.

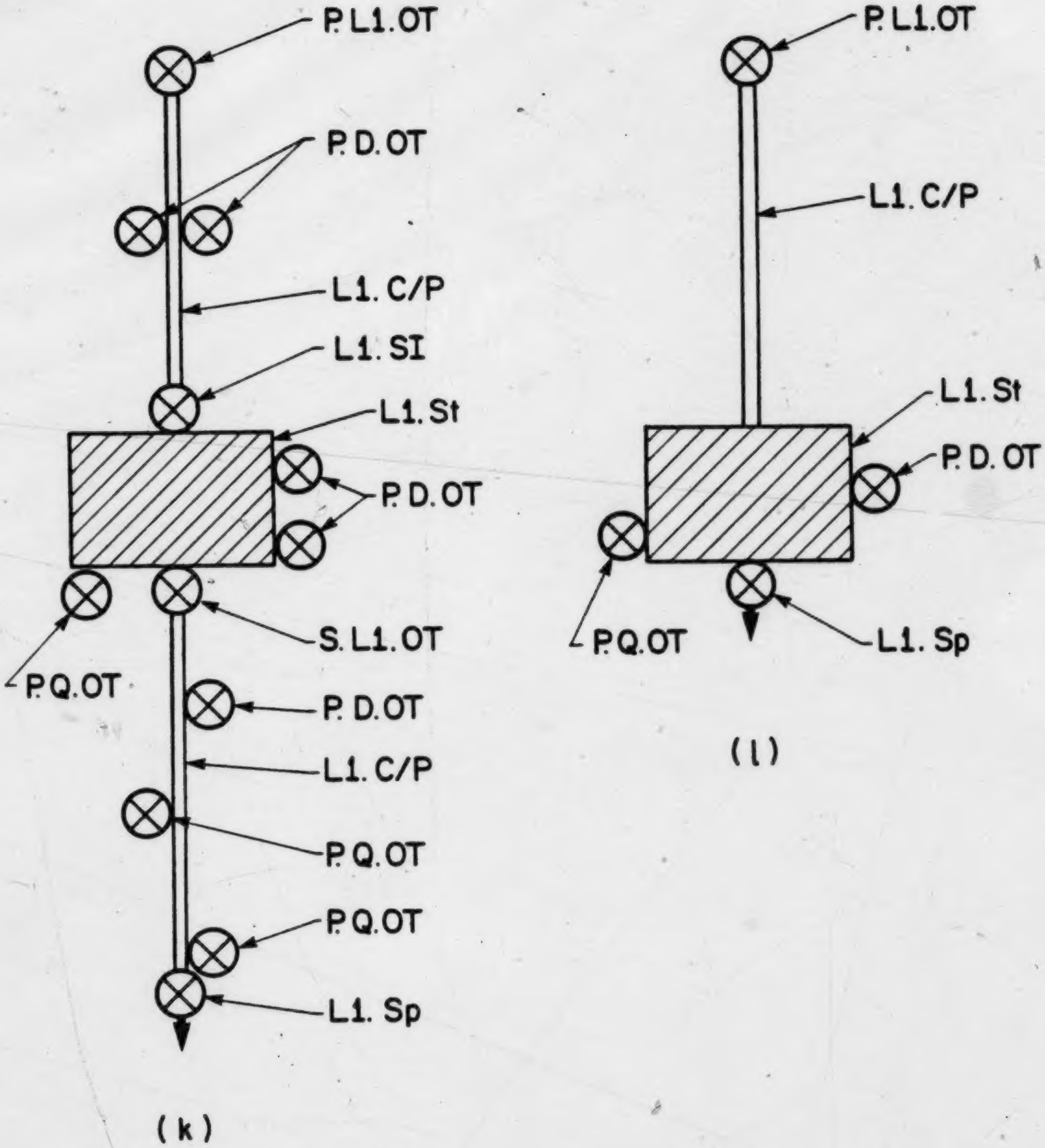

P. L1. OT
P. D. OT
L1. C/P
L1. SI
L1. St
P. D. OT
S. L1. OT
P. Q. OT
P. D. OT
L1. C/P
P. Q. OT
P. Q. OT
L1. Sp
(k)
P. L1. OT
L1. C/P
L1. St
P. D. OT
P. Q. OT
L1. Sp
(l)

Figure 5.3 (k) and (l) Examples of L1 conveyance subsystems.

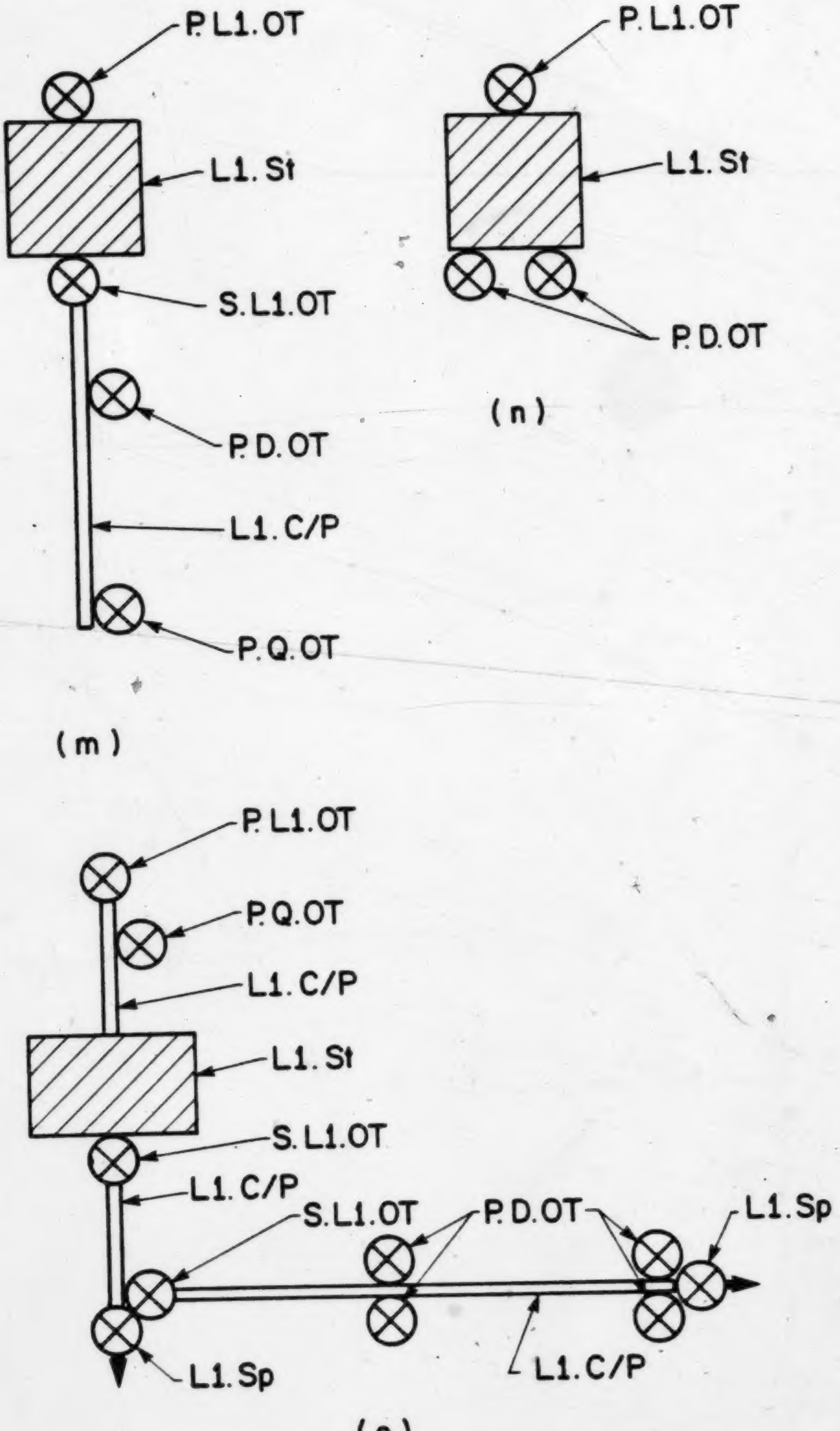

P. L1. OT
L1. St
S. L1. OT
P. D. OT
L1. C/P
P. Q. OT
(m)
P. L1. OT
L1. St
P. D. OT
(n)
P. L1. OT
P. Q. OT
L1. C/P
L1. St
S. L1. OT
L1. C/P
S. L1. OT
P. D. OT
L1. Sp
L1. Sp
L1. C/P
(o)

Figure 5.3 (m) - (o) Examples of L1 conveyance subsystems.

level ponds. The subsystems supplied by these storage facilities would likely be closed pipeline systems. Figure 5.3(o) illustrates a case where both applications of secondary L1 off-takes are used.

5.6 Second and Higher Order Lateral Subsystems

Second and higher order lateral conveyance subsystems, L2, L3, ..., LN, facilitate the delivery of water to the primary off-takes of two or more lateral units one order lower than the lateral subsystem delivering the water or to the primary off-takes of a single lateral unit one order lower than the lateral delivering the water and at least one primary off-take of a unit two or more orders lower than the lateral delivering the water. In all other respects, second and higher order lateral subsystems are similar to first order, L1, laterals.

Examples of higher order lateral subsystems are illustrated in Figures 5.4(a) to (h). Figures 5.4(a) to (d) provide simple but typical examples of higher order lateral subsystems. Note that examples (b) and (c) employ division structures. Figures 5.4(e) and (g) should be contrasted. The canal in Figure 5.4(e) is an L4 throughout its length. Note the affect of storage facilities in Figure 5.4(g). Figure 5.4(f) illustrates a typical L4 subsystem which incorporates a single storage facility. Figure 5.4(h) illustrates an L4 subsystem which employs three similarly classified storage facilities.

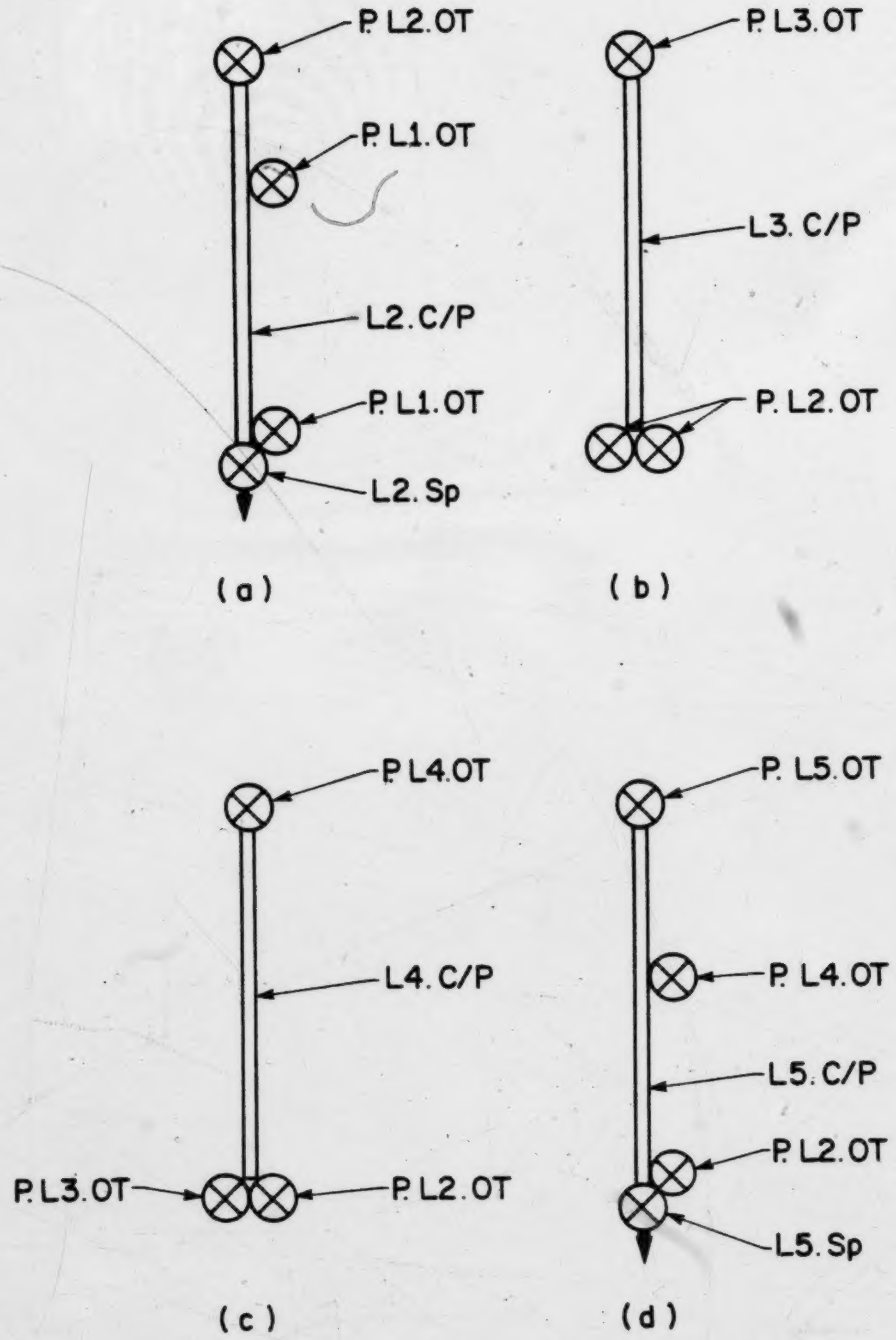
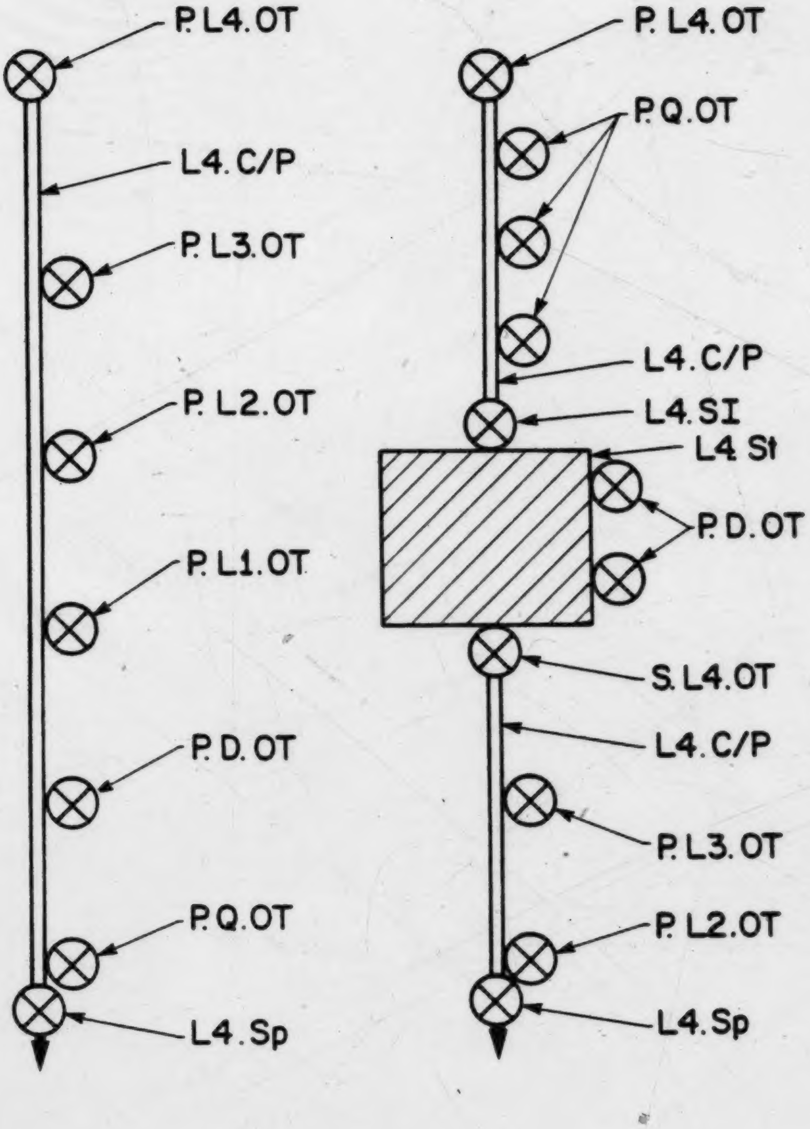

P. L2.OT
P. L1.OT
L2. C/P
P. L1.OT
L2. Sp
(a)
P. L3.OT
L3. C/P
P. L2.OT
(b)
P. L4.OT
L4. C/P
P. L3.OT
P. L2.OT
(c)
P. L5.OT
P. L4.OT
L5. C/P
P. L2.OT
L5. Sp
(d)

Figure 5.4 (a) - (d) Examples of higher order lateral conveyance subsystems.


P. L4. OT
L4. C/P
P. L3. OT
P. L2. OT
P. L1. OT
P. D. OT
P. Q. OT
L4. Sp
P. L4. OT
P. Q. OT
L4. C/P
L4. SI
L4. St
P. D. OT
S. L4. OT
L4. C/P
P. L3. OT
P. L2. OT
L4. Sp

(e) (f)

Figure 5.4 (e) and (f) Examples of higher order lateral conveyance subsystems.

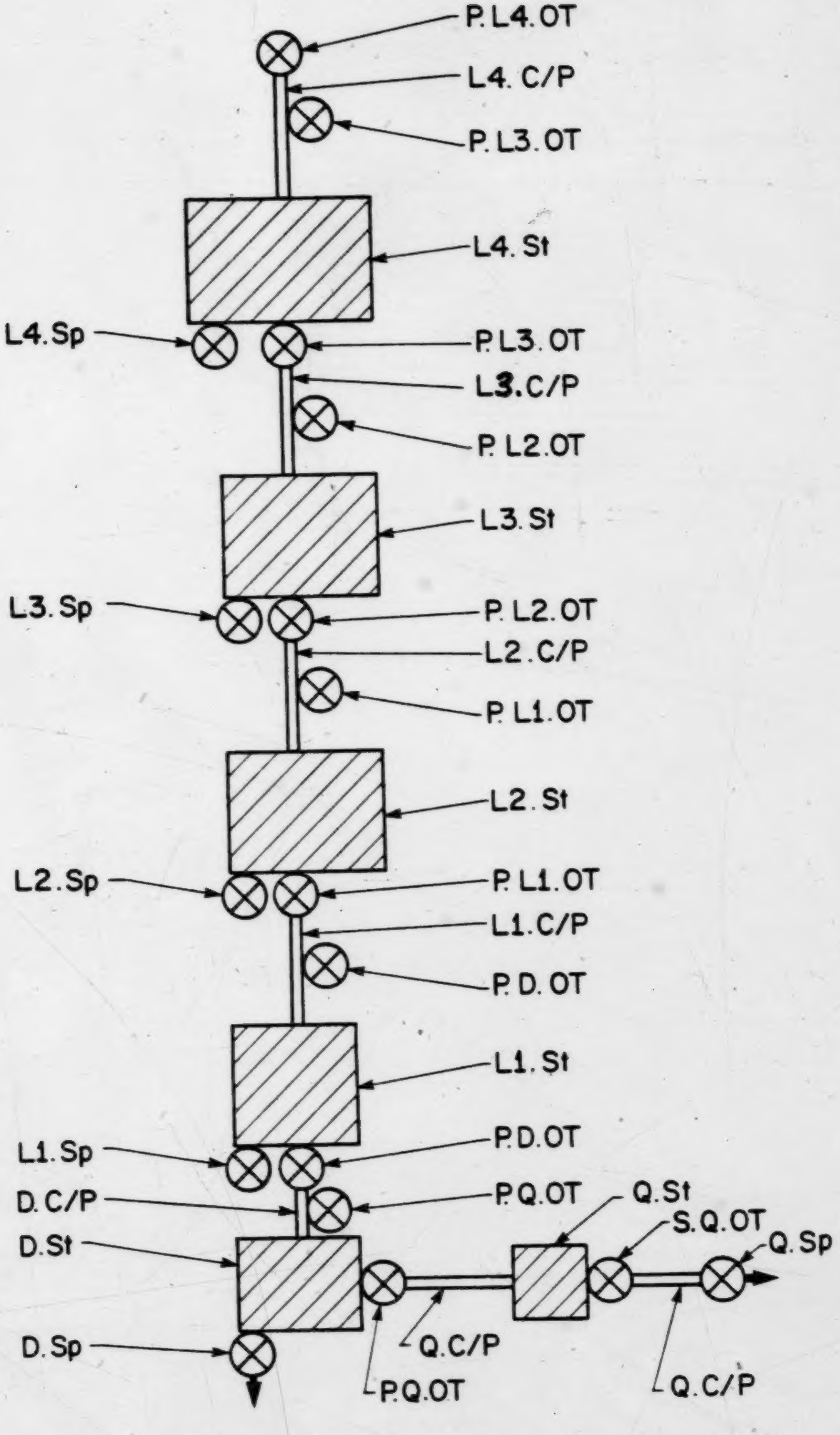

P. L4. OT
L4. C/P
P. L3. OT
L4. St
L4. Sp
P. L3. OT
L3. C/P
P. L2. OT
L3. St
L3. Sp
P. L2. OT
L2. C/P
P. L1. OT
L2. St
L2. Sp
P. L1. OT
L1. C/P
P. D. OT
L1. St
L1. Sp
P. D. OT
D. C/P
P. Q. OT
Q. St
S. Q. OT
Q. Sp
D. St
D. Sp
Q. C/P
P. Q. OT
Q. C/P

Figure 5.4(g) Example of a higher order lateral conveyance subsystem.

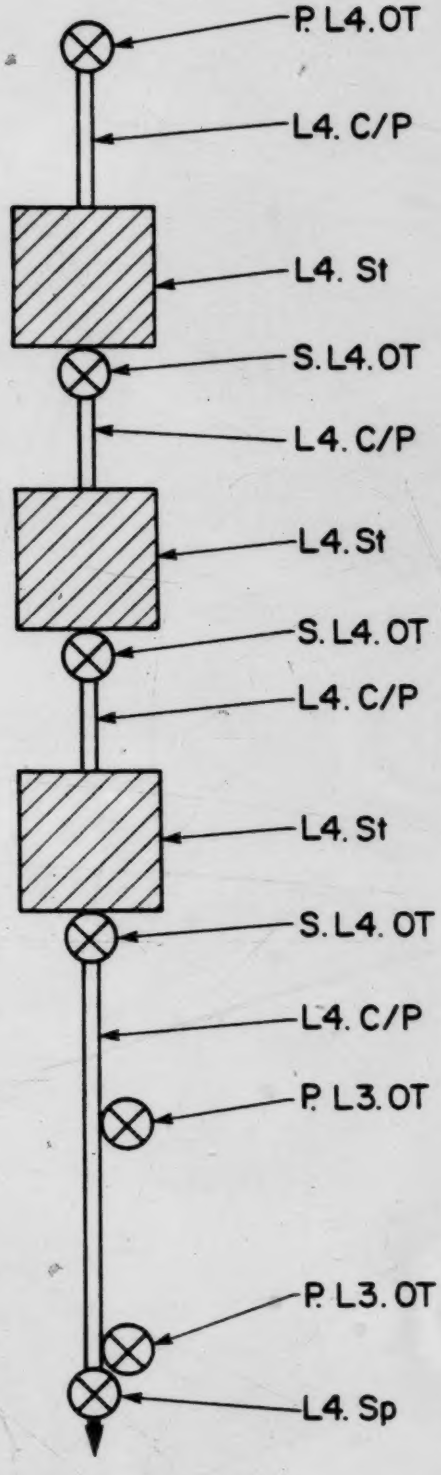

P. L4. OT
L4. C/P
L4. St
S. L4. OT
L4. C/P
L4. St
S. L4. OT
L4. C/P
L4. St
S. L4. OT
L4. C/P
P. L3. OT
P. L3. OT
L4. Sp

Figure 5.4(h) Example of a higher order lateral conveyance subsystem.

5.7 Main Canal/Pipeline Concept

The conveyance subsystem, which receives its water through a single diversion structure from the only source of water for all downstream water users, is called the main canal/pipeline subsystem. The main subsystem may be a quaternary, distributary or a lateral subsystem, depending on the particular conveyance system being examined. The diversion structure which supplies the main canal/pipeline is called the headwork diversion structure and the water source is called the primary water supply.

5.8 Management Considerations

An entire irrigation conveyance system is normally managed by a minimum of two cooperating management organizations one of which is usually the land owner and/or manager. This fact has often confused the identification of irrigation conveyance subsystems because it is generally recognized that each management organization will determine conveyance system design, nature and quality of construction, maintenance procedures and operational procedures for that portion of the conveyance system within its ownership and/or control. For this reason many investigators have attempted to include the management relationship in their terminology. Unfortunately, this has resulted in a proliferation of inexact descriptors such as farm canal, district canal, field canal, etc. whose meaning may be remarkably precise locally but vary considerably with region. These difficulties may be avoided if the terminology just presented is always used and the ownership/management status is simply provided by way of additional information as opposed to the introduction of new terminology.

5.9 Terminology Legend

5.9.1 Quaternary Subsystems

1. P.Q.OT - primary quaternary off-take
2. Q.U. - quaternary unit
3. Q.C/P - quaternary canal and/or pipeline
4. Q.Sp - quaternary spill
5. Q.SI - quaternary storage inflow structure
6. Q.St - quaternary storage facility
7. S.Q.OT - secondary quaternary off-take
8. Q.Sp - quaternary spill
9. Q.DI - quaternary drainage inflow

5.9.2 Distributary Subsystems

1. P.D.OT - primary distributary off-take
2. D.U - distributary unit
3. D.C/P - distributary canal and/or pipeline
4. D.Sp - distributary spill
5. D.SI - distributary storage inflow structure
6. D.St - distributary storage facility
7. S.D.OT - secondary distributary off-take
8. D.Sp - distributary spill
9. D.DI - distributary drainage inflow

5.9.3 Lateral Subsystems

1. P.L1.OT - primary L1 lateral off-take
2. L1.U - L1 unit
3. L1.C/P - L1 canal and/or pipeline
4. L1.Sp - L1 spill
5. L1.SI - L1 storage inflow
6. L1.St - L1 storage
7. S.L1.OT - secondary L1 off-take
8. L1.Sp - L1 spill
9. L1.DI - L1 drainage inflow

5.10 Discussion

The terminology presented offers several advantages:

1. All components which comprise irrigation conveyance systems may be identified with respect to their relationship to the irrigation water demand they are intended to satisfy. Those components not specifically named in the terminology presented - for example, depth-control structures may be identified quite easily using the subsystem classification as a leading adjective as follows; quaternary depth-control structure, L1-culvert and distributary depth-control structure.
2. Within individual conveyance systems, similarly classified components can be expected to exhibit a degree of physical similarity. This is more true for the components nearer the primary off-takes of the subsystems.

3. The subsystem classification, by indicating the relationship to the irrigation water demand being satisfied, provides an approximate index of the complexity associated with operating the conveyance system to make water deliveries. This is certainly true for individual conveyance systems.
4. Considering advantages 2 and 3, the subsystem classification provides a rational means with which to identify similar portions of irrigation conveyance systems for purposes of comparative performance evaluation studies.
5. The subsystem classification, combined with the system for classifying irrigation conveyance system components presented in Chapter 4, provide a terminology which permits concise relevant discussion of irrigation conveyance system design and operations.

It should be noted that the terminology presented in this Chapter for describing irrigation conveyance systems does not necessarily replace or render redundant other terminology based on subsystem relationship to primary water supply. These techniques are usually used to identify, locate and name the individual subsystems and their components. An example of a terminology based on subsystem relationship to primary water supply is the Alberta Department of Agriculture Method described in Appendix C. Examination of this technique will reveal that similarly classified subsystems may be very different both in terms of the water demand they satisfy and in their physical and operational characteristics. Terminologies based on subsystem relationships to water demand and to primary water supply may be considered complimentary.

CHAPTER 6

OPERATION OF IRRIGATION CONVEYANCE SYSTEMS

6.1 Introduction

The objective of irrigation conveyance system operations is to deliver water from a source (or sources) to satisfy farm water demands. The operation of an irrigation conveyance system is concerned with the coordinated control of systems of conveyance system components to achieve this end.

The operation of individual conveyance systems is a complex function of the type of farm water demand being satisfied, the type(s) of conveyance system components and their configuration. The broad range of operational possibilities available may be described in terms of their orientation to water supply or water demand and by the type of control used to operate the system.

6.2 Orientation of Irrigation Conveyance System Operations

Canals or pipeline systems may be orientated to demand or supply operations or combinations of both, Bulyaski (1979). Demand orientation implies that water is removed from the canal in the downstream direction. Supply orientation implies that water is added to the canal in the downstream direction. Combination demand and supply orientation implies that water may be taken from and added to the canal in the downstream direction. Most irrigation conveyance systems are demand orientated. Relatively few are combination orientated.

6.3 Type of Control

Control of irrigation conveyance system operations may be one of three types: upstream control, downstream control, and dynamic.

6.3.1 Upstream Control

6.3.1.1 General Description

Upstream control is described by Cunge and Woolhiser (1975), Nelson (1980), Dedrick and Zimbelman (1981), Kraatz and Mahajan (1975), Burt and Lord (1981), and Burt (1982). When upstream control is used, sufficient water is diverted into the irrigation conveyance system to satisfy downstream farm water demands and any distributed water losses (seepage, for example) from the canal. Diversion structures, through which water demands are satisfied, are operated when the water supply is available. The availability of water supply at the diversion structures is determined by the simultaneous occurrence of adequate water depth at the structure and flow rate past the structure. Water depth may be controlled using depth-control structures. Water not delivered or lost en route is spilled from the system. When upstream control is used, the operation of all diversion and depth-control structures are locally coordinated. Dewey and Madsen (1976), in their description of early operations of the California Aquaduct, aptly labelled upstream control a "serial gate operation".

6.3.1.2 Examples of Manual Upstream Control

Numerous examples of conveyance systems operated using manual upstream control may be found in the literature. The operation and control of irrigation conveyance systems in South India are described by Wade (1981) and Kraatz and Mahajan (1975); in the Indus Basin of Pakistan by Johnson et al. (1978), and Trout and Bowers (1981); in Taiwan by Abel (1976); in Sri Lanka by Moore (1981); in the United States by Harding (1917), Hill (1952), USBR (1951), the ASCE (1980), and Clemmens and Dedrick (1984); and, in Canada by Manz (1984(a)). The work reported by Manz is presented.

The particular distributary canal to be considered is illustrated in Figure 6.1. It is an example of a manually operated distributary canal which employs upstream control is described. The canal is typical of those found in irrigation districts in Southern Alberta, Canada. The primary distributary off-take supplying the canal receives water from a lateral canal. The distributary off-take is a Type 5 diversion structure. The quaternary off-takes are Type 5 diversion structures and the two distributary depth-control structures are Type 1 depth-control structures. The canal is unlined and therefore subject to seepage losses and infestation by vegetation. The canal terminates in a drop structure.

The farm water demand being satisfied may be described as being collaboratively controlled unlimited frequency, collaboratively controlled limited rate and collaboratively controlled unlimited

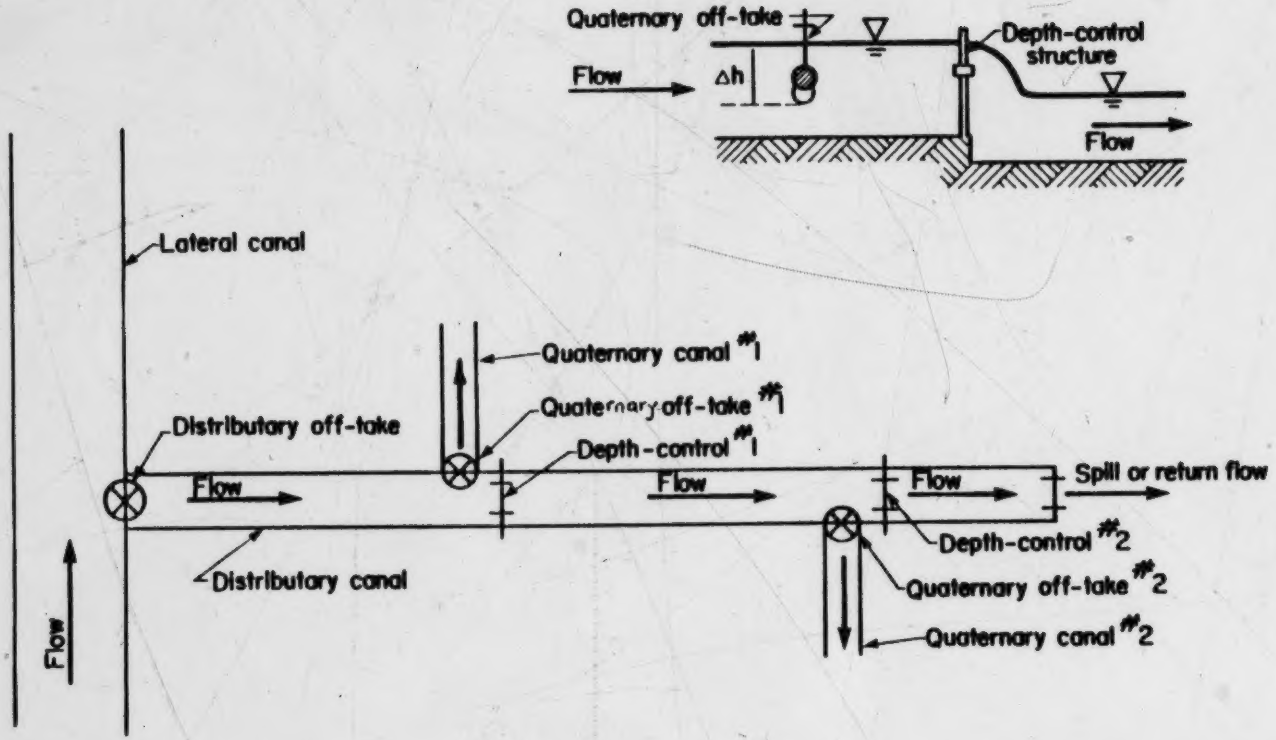

Quaternary off-take
Flow
Δh
Depth-control structure
Flow
Lateral canal
Quaternary canal #1
Quaternary off-take #1
Distributary off-take
Depth-control #1
Flow
Flow
Flow
Spill or return flow
Depth-control #2
Distributary canal
Quaternary off-take #2
Flow
Quaternary canal #2

Figure 6.1 Example of distributary canal operation using manual upstream control.

duration. This corresponds to the Repogle and Merriam Type 5, restricted arranged schedule.

Operation of the canal proceeds as follows:

1. The canal operator (ditchrider) receives notice from the farmers that water is required.
2. The canal operator estimates the farmers' actual water requirements.
3. The canal operator estimates the magnitude of distributed losses which will occur while the water is transferred from the distributary off-take to the quaternary off-takes.
4. The canal operator estimates the magnitude of the required release into the distributary canal.
5. The canal operator operates the primary distributary off-take and diverts water into the distributary canal. The actual amount diverted into the canal will likely differ from his objective due to imprecise distributary off-take operations.
6. The canal operator will operate the depth-control structures immediately downstream of the quaternary off-takes to ensure that there is adequate depth of water in the canal to operate the quaternary off-takes. (e.g. He will raise the height of the weir if it is a weir-type depth-control structure.)
   The water actually released into the canal will be reduced by unknown distributed loss rates, which are a function of the canal hydraulic and hydrologic characteristics at the time the canal is operated and by the backwater effects caused by the operation of the depth-control structure. (See Appendix E.)

7. When the operator is sure that steady-state conditions are achieved at the quaternary off-takes, he will operate them and make the delivery to the farmers.
8. The farmers are ready to accept this delivery and extract their water application needs from the delivery.
9. Water delivered to the farmers in excess of their needs may be spilled by the farmers into some available drainage system.
10. Water not delivered to the farmers is allowed to spill down the distributary canal. As the spilled water passes down the distributary canal, its volume is reduced by additional distributed losses incurred while enroute.
11. Spilled water which reaches the end of the canal moves into an available drainage system and is called surface return flow.

During these operations the procedures the canal operator may use various measures to reduce excessive spill. These may be as follows:

12. After a period of time, the farmers may complain to the canal operator of excess delivery. (With the use of sprinkler irrigations systems, excess deliveries are preferred to shortfalls.)
13. The canal operator will assess the farmers' complaints and reduce delivery to them.
14. Additional distributary spill will occur.
15. The canal operator assesses the spill. (Usually, he assesses the volume of surface return flow.)
16. The canal operator reduces the diversion into the distributary canal by operating the primary distributary off-take.

17. After a period of time, the canal operator will reassess the spill and may repeat 15.

If the flow through the distributary off-take remains constant and if the farmer's water consumption rate continues unchanged, the delivery will continue until the farmer requests that it be stopped.

6.3.1.3 Examples of Automatic Upstream Control

Specific examples describing the use of automatic upstream control for operating irrigation canals are discussed by Bulyaski (1970), Bulyaski (1977), Nelson (1980), Dedrick and Zimbelman (1981), and Kraatz and Mahajan (1975). The example presented is similar to one presented by Nelson (1980) and Dedrick and Zimbelman (1981) and is illustrated in Figure 6.2. The primary L1 off-take receives water from an L2 lateral. The L1 off-take is a Type 14 diversion structure. The primary distributary off-takes are Type 5 diversion structures. There are two Type 18 depth-control structures and one Type 22 depth-control structure controlling the water level adjacent the distributary off-takes. The L1 lateral canal is unlined and therefore subject to seepage losses and vegetation infestations. Only the upstream portion of the lateral is shown in Figure 6.2.

The water demand the canal is satisfying may be described as collaboratively controlled unlimited frequency, collaboratively controlled limited rate and collaboratively controlled unlimited duration. Note that these water demands are similar to the Repogle and Merriam Type 5, restricted arranged schedule.

The operation of the canal proceeds as follows:

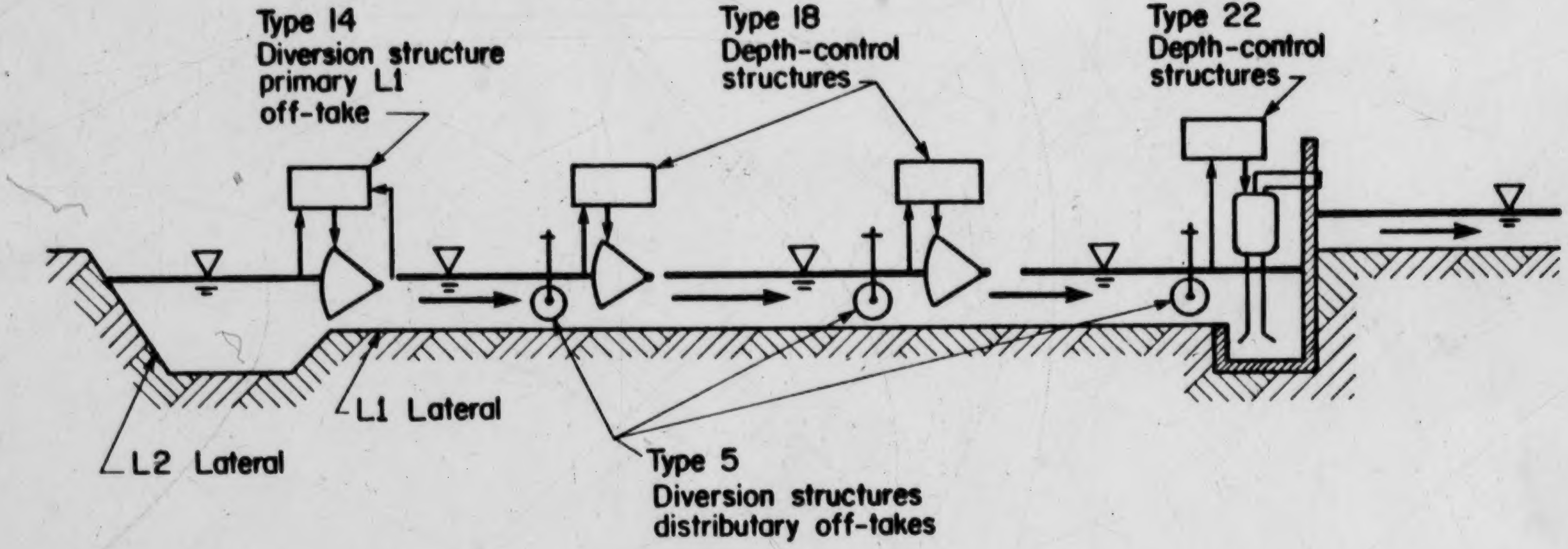

Type 14
Diversion structure
primary L1
off-take
Type 18
Depth-control
structures
Type 22
Depth-control
structures
L1 Lateral
L2 Lateral
Type 5
Diversion structures
distributary off-takes

Figure 6.2 Example of L1 lateral canal operation using automatic upstream control.

1. The canal operator receives water orders from the canal operators who operate the primary distributary off-takes; or, if he operates the distributary off-takes himself, he estimates the water demands.
2. The canal operator estimates the magnitude of distributed losses which will occur while the water is transferred from the primary L1 off-takes to the primary distributary off-takes.
3. The canal operator estimates the magnitude of the required release into the L1 canal.
4. The canal operator operates the primary L1 off-take by providing the controller with his target diversion requirements.
5. The controller compares the target diversion requirements to the current diversion and adjusts the L1 off-take gate opening till target and actual diversion match.
6. The water actually released into the canal will be reduced by essentially unknown distributed losses.
7. The operator will establish target upstream water depth values for each of the L1 depth-control structures.
8. The controller of each of the L1 depth-control structures will compare the actual upstream water depth to the target values and adjust the gate openings until target and actual water depths match.
9. When the operator is reasonably certain that the flow has reached the depth-control structures which were just altered,

he will divert water into the distributary canals by manually operating the primary distributary off-takes.

10. After each distributary off-take is adjusted, a period of unsteady flow downstream of each operated distributary off-take will occur. The impact of the associated flow rate fluctuation on water depth will be minimized by the automatically operated depth-control structures.
11. Water diverted into the L1 lateral in excess of that required may be spilled from the canal into an internal storage facility or into some available drainage system where it becomes surface return flow.
12. After a period of time the canal operator may assess the rate of spill and reduce the target diversion through the primary L1 off-take.

Steps 1 through 13 may be repeated each day.

6.3.2 Downstream Control

6.3.2.1 General Description

Downstream control is described by Cunge and Woolhiser (1975), Nelson (1980), Dedrick and Zimbelman (1981), Kraatz and Mahajan (1975), Bulyaski (1979), Burt and Lord (1981), and Burt (1982). When downstream control is used, sufficient water is available within the conveyance system for the immediate operation of downstream diversion structures. Water is diverted into the conveyance in response to the depletion of downstream water supplies. Ideally, the water supplies are such that they are never depleted to the extent that the diversions are inhibited. When the downstream diversion operations are completed, water may continue to be released into the conveyance system until internal water supplies are replenished. When downstream control is used, all diversion and depth-control structures are locally coordinated.

6.3.2.2 Example of Manual Downstream Control

Manual downstream control of open channel irrigation conveyance systems is infrequently used. However, closed pipeline systems are frequently downstream controlled and an example of such a conveyance system is described.

The closed distributary pipeline to be considered is shown in Figure 6.3. The primary distributary off-take and the primary quaternary off-takes are each Type 1 diversion structures. The primary distributary off-take receives water from an L1 canal.

The farm water demand being satisfied may be described as farmer controlled unlimited frequency, farmer controlled limited rate, and farmer controlled unlimited duration. This type of farm water demand corresponds to the Repogle and Merriam Type 2, limited rate demand schedule.

The operation of the pipeline is apparent. Whenever a farmer requires water he simply opens the off-take and starts pumping. Sufficient water will enter the pipeline as long as the distributary off-take is opened, submerged and clear of debris.

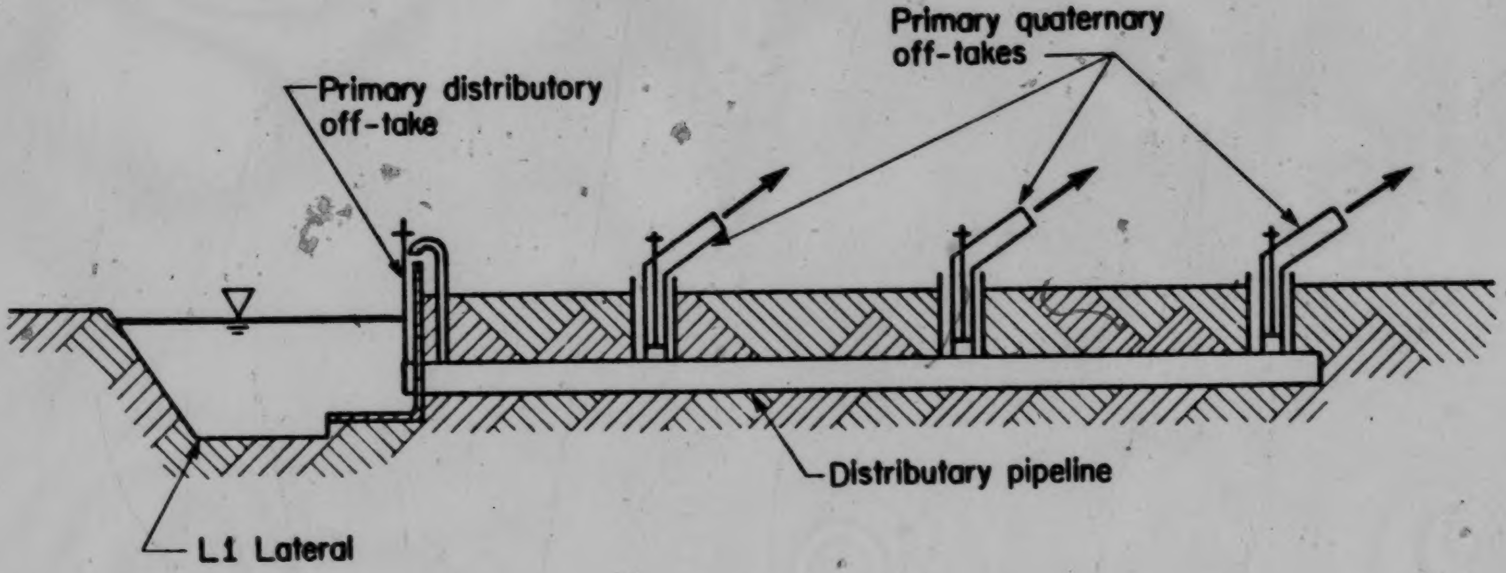

Primary quaternary off-takes
Primary distributory off-take
Distributary pipeline
L1 Lateral

Figure 6.3 Example of distributary pipeline operation using manual downstream control.

6.3.2.3 Example of Automatic Downstream Control

Specific examples describing the use of automatic downstream control for operating irrigation canals is discussed by Bulyaski (1979), Merriam (1977), Cunge and Woolhiser (1975), Kraatz and Mahajan (1975), Ehler (1979), Nelson (1980), Dedrick and Zimbelman (1981), Burt (1982), and Zimbelman and Bedworth (1983). The example presented is similar to one presented by Nelson (1980) and Dedrick and Zimbelman (1981) and is illustrated in Figure 6.4. The primary L1 off-take receives water from an L2 storage facility (a balancing pond or regulating reservoir). The L1 off-take is a Type 14 diversion structure. The primary distributary off-takes are Type 5 diversion structures. Secondary L1 off-takes number one and two are Type 6 diversion structures; and, secondary off-take number three is a Type 2 diversion structure. The L1 lateral canal is lined with concrete and is therefore not subject to seepage losses and vegetation infestation. Only the upstream portion of the lateral is shown in Figure 6.4.

The water demand the canal is satisfying may be described as user controlled unlimited frequency, user controlled limited rate, and user controlled unlimited duration. Note that this is similar to Repogle and Merriam Type 2, limited rate demand schedule except that farm water demand is replaced with user or distributary water demand.

In the example presented each of the L1 off-takes receive water depth information from immediately upstream of the following L1 off-take. (Other flow condition information may be used as is described in literature cited.) The controllers receive the water depth

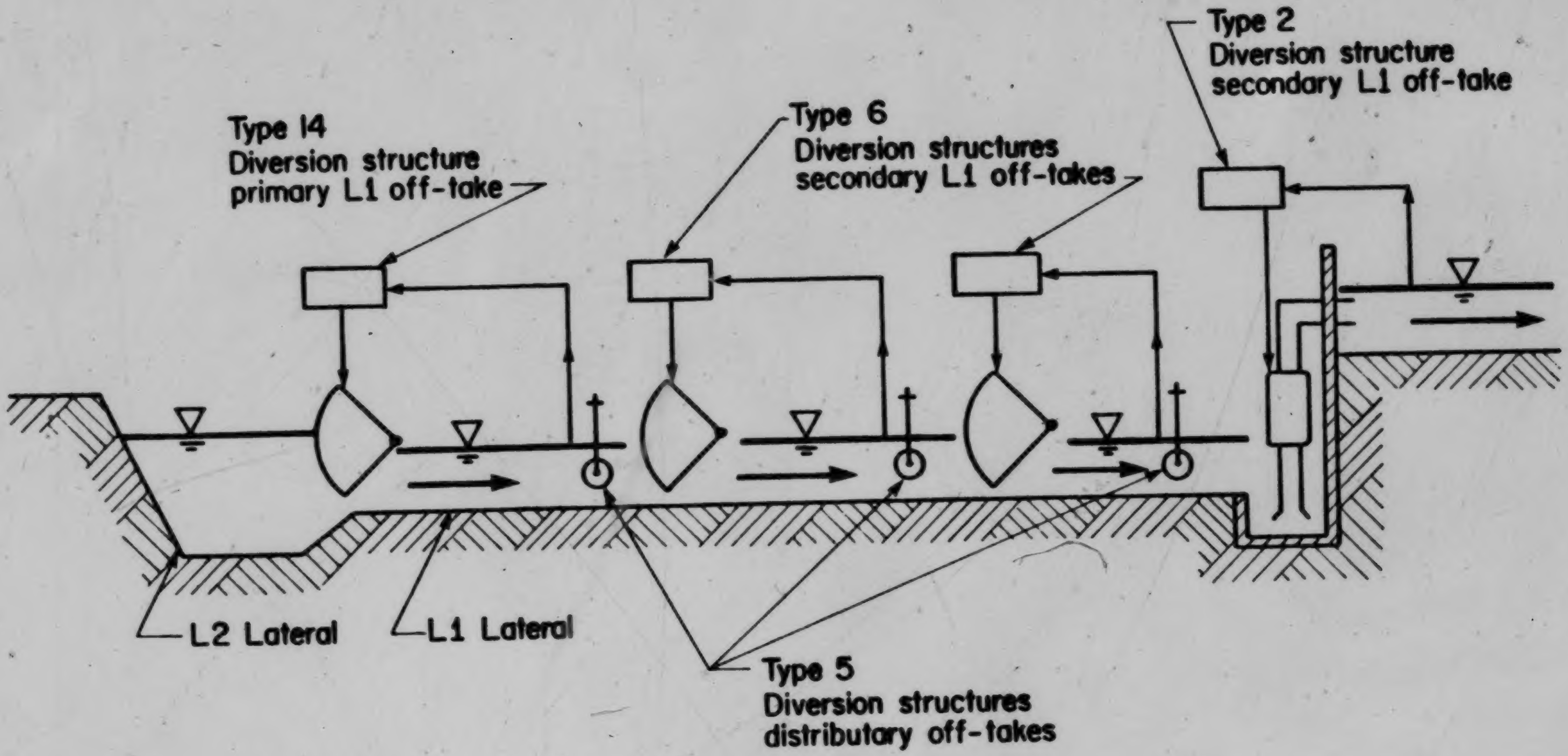

Type 14
Diversion structure
primary L1 off-take
Type 6
Diversion structures
secondary L1 off-takes
Type 2
Diversion structure
secondary L1 off-take
L2 Lateral
L1 Lateral
Type 5
Diversion structures
distributary off-takes

Figure 6.4 Example of L1 lateral operation using automatic downstream control.

information and convert this to a gate opening (depending on the specific characteristics of the control algorithm), thus increasing or decreasing the release of water into the downstream reach. The water level in the downstream reach may fluctuate either because the primary distributary off-take in the downstream reach is diverting water or because the next L1 off-take is modifying its water release to its downstream reach. For whatever reason the water levels are fluctuating, the L1 off-take adjustments will continue upstream until the diversion structure at the canal inlet is operated.

6.3.3 Dynamic Regulation

6.3.3.1 General Description

Dynamic regulation is identified as a type of conveyance system control by Kraatz and Mahajan (1975), Burt (1982), and Burt and Lord (1981). Dynamic regulation implies that some or all of the diversion and depth-control structures are system co-ordinated. Information on water supply conditions, the status of diversion and depth-control structures together with information on planned diversion operations is assembled and analyzed to generate operating rules which will satisfy the diversion requirements and still maintain the conveyance system in a secure condition that will permit "prompt" satisfaction of future water diversion requirements. Kraatz and Mahajan (1975) define dynamic regulation as follows: "Dynamic regulation may be defined as a means of seeking and implementing the regulation optimum in relation to a set of conditions existing at a given moment and in line with a given number of criteria. In this context the set of conditions refers to water level, flows, gate positions, valve openings, etc., and the criteria may be consumption forecasts, physical and economical constraints and safety margins." Burt (1982) indicates that: "Dynamic regulation techniques differ with respect to the type and quantity of information gathered, the frequency with which the information is gathered, the frequency with which structures are operated and the flexibility provided the users."

6.3.3.2 Examples of Dynamic Regulation

Two prominent and frequently described examples of conveyance systems operated using dynamic regulation are the California Aquaduct and the Canal de Provence in Southern France. Frederiksen (1969) describes the design and operation of the California Aquaduct which employed what is called the "control volume concept". Dewey and Madsen (1976) describe the specific operational characteristics of the California Aquaduct and the development of its current operational system, a modified control volume concept called "time gate operation". The various factors which affect the control of the California Aquaduct are described by Devries and Amorocho (1973). Burt and Lord (1981) describe the current operation of the California Aquaduct as follows: "The California Aqueduct is essentially a series of level-topped pools. Flow changes are initiated by knowing in advance what the users want in the way of water deliveries. A computer calculates the anticipated required flow rates in each section of canal. A second program calculates gate openings, simulates the canal operation with these spacings, and then readjusts the calculated gate openings. These calculated gate openings are used during the day and readjusted periodically.

The California Aqueduct uses timed gate operation. Originally all check gates were operated simultaneously, but this created large hydraulic transient flow problems. With timed gate operation, a variety of gate movements can be made to suit the operating situation. One or all of the gates at each check structure can be operated and timed to start and stop several times before reaching the predetermined gate position. Because water flows from all upstream pools into all downstream pools almost simultaneously, the distance from the ultimate water source to the Aqueduct diversion point has no appreciable effect on the time needed to meet the delivery change."

The operation of the Canal de Provence in Southern France is described by Kraatz and Mahajan (1975), Cunge and Woolhiser (1975), Burt and Lord (1981), and Burt (1982). Kraatz and Mahajan (1975) describe the operation of the Canal de Provence as follows:

"One of the most recent and most sophisticated of flow control systems is the one developed by the Société du Canal de Provence for the "Verdon Canal Extension" of the Canal de Provence, France. The Verdon canal network, consisting of approximately 60 km of main and branch canals, conveys water for multiple uses (irrigation, industry and urban water supply) in the Region of Aix-en-Provence. The system is also of considerable complexity with regard to varying types of canal (size, grade, lining), type and age of structures, tunnels, pressure pipes, pumping stations, reservoirs, etc. The nominal head discharge is 6.5 m^3/s; water is delivered on demand by means of pressure pipe systems.

Because of this complexity and a number of other considerations special attention was paid to the operational system to be adopted for the project. Efforts were made to find out how best to make use of the network and electrical installations which already existed. The studies resulted in the development of the control system known as dynamic regulation.

Dynamic regulation is based on the simultaneous sensing of water levels, by means of float devices or electric probes, at a large number of points within the network. The system cited includes a communication network for the transmission of data (on water levels, flows, gate positions, etc.) to and from a general control centre (situated in the town of Tholonet). In order to collect, check and interpret these data and process the orders to be given to the various gates, pumps, etc. (by application of a mathematical model) and so as to give these orders at the right time and check their proper execution, the installation of a computer, (working on a real-time basis) was found to be indispensable. The choice fell on an IBM 1800. It was necessary, furthermore, to develop computer programmes for the implementation of regulation principles and to adapt them to the characteristics of the equipment to be operated.

The system has been subjected to various tests to determine the hydrodynamic properties of the network while maintaining canal banks and other engineering structures in their original form as far as possible. The tests have also helped in appraising the overall dependability of the regulation system by simulating breakdown and faulting of demand schedules, etc. This dynamic control system has been operating successfully since 1971."

6.4 Discussion

Though the control of irrigation conveyance systems may be considered to be one of three types, (manually and automatically controlled sub-types withstanding), there still exists considerable scope for operation variability within the range of possible "styles" of the control type selected. The style of control would be a function of a variety of factors including the nature of farm water demand, communication systems, mobility of operators, economics of agriculture practised, skill of operators, local traditions, and a variety of other cultural and social influences. As well, modifications in operational style are often required within similarly configured systems to accommodate similarly classified but physically different components. Each variation of similarly classified components normally would require special operational considerations.

Another factor, which further complicates the consideration of irrigation conveyance system operations, is the frequently encountered condition where several methods of operational control are used to operate the same system or subsystem. Though a greater degree of operational heterogeneity would be expected in older irrigation projects, it would not be unreasonable to expect several operational methods to be incorporated in the design of new projects.

CHAPTER 7

PERFORMANCE EVALUATION OF IRRIGATION CONVEYANCE SYSTEMS

7.1 Introduction

As stated previously, the function of irrigation conveyance systems is to deliver water from a source, (or sources), to satisfy farm water demands. It is recognized that costs of operating the system should be kept as low as reasonably possible and that the water resources available for irrigation purposes (developed or otherwise) are limited. It is therefore apparent that the parameters with which irrigation conveyance system operations may be evaluated are:

1. economy of operation,
2. satisfaction of farm water demand, and
3. efficiency of water conveyance.

7.2 Economy of Operation

Economy of operation of an irrigation conveyance system is related to the physical and environmental characteristics of the conveyance system and the operational procedures adopted. (It is worth noting that the operational procedures adopted will be a function of the type of farm water demand to be satisfied, the efficiency of water conveyance that will be tolerated and a variety of other management policies.) Economy of operation is quantifiable in the sense that most, if not all, operational procedures, can be "costed" once their specific nature is known. The benefits to be accrued by the adoption of detailed cost accounting of irrigation project operations is discussed by Hill (1952). Shipley (1970) discusses how the study of operational methods used in the Salt River Project in Arizona provided the necessary cost information to justify the introduction of apparently costly remote controlled systems for operating a number of hydraulic structures.

7.3 Satisfaction of Farm Water Demand

Satisfaction of farm water demand relates to the question: "How well is the water delivery process being performed - from the farmers' perspective?" This question is considered significant enough that it is included in the list of high priority research objectives identified by the ASCE Task Committee on the status of Irrigation and Drainage Research of the Committee on Research of the Irrigation and Drainage Division, ASCE (1984). The Task Committee has tacitly recognized that there is a direct relationship between the physical, environmental, and operational characteristics of an irrigation conveyance system and the type of farm water demand it is capable of satisfying. Clemmens and Dedrick (1984) report research which quantifies fluctuations in water deliveries in terms of its one-half hour coefficient of variation and attempted to relate this quantity to farm water application efficiency.

It is suggested that rather than using the one-half hour coefficient of variation of farm water delivery as the quantitative measurement of satisfaction of farm water demand, that upper and lower flow rate (or volume) tolerance limits be placed on the farm water demand and that water deliveries which fall within these limits be considered satisfactory. The coefficient of variation of farm water delivery is relevant only to diversions supplying surface irrigation directly. The percentage of the time the water delivery satisfied farm water demands would then be a measure of farm water demand satisfaction. The tolerance limits imposed on farm water demand would be a reflection of the specific nature of the farm water demand being satisfied; that is, crop water requirements, application system, physical and operational characteristics of on-farm irrigation conveyance system, climate and a variety of other socio-economic factors.

Tolerance limits of the type applicable to farm water demands would also be useful for evaluating the satisfaction of all water demands - distributary, L1, L2, etc. - throughout an irrigation conveyance system.

7.4 Efficiency of Water Conveyance

7.4.1 General Description

The desire to evaluate the effectiveness of the numerous water management activities which occur within irrigation projects have resulted in the development and use of a variety of performance descriptors which may collectively be called "irrigation efficiencies". Many agencies, professional societies, and individuals representing most countries in the world, where irrigation is practiced, have participated in the development of irrigation efficiency concepts. Significant publications which discuss irrigation efficiency concepts include: ICID (1978), Bos and Nugteren (1978), Bos (1979) and (1980), Jensen (1967),

(1973) and (1977), Jensen et al. (1980), Shmeuli (1973), Kruse and Heerman (1977), Israelsen and Hansen (1962), and Burman et al. (1980).

In general irrigation efficiencies are determined by isolating the system or subsystem of interest and then performing a water balance which may be more or less detailed. A schematic of a generalized system is shown in Figure 7.1. The efficiency is defined as the ratio of the quantity of water available for "beneficial" use or "beneficially" consumed after the process is completed to the quantity of water made available at the start of the process. In equation form:

$$\text{Efficiency} = \frac{\text{Quantity of water available for beneficial use or beneficially consumed after process is completed.}}{\text{Quantity of water made available at start of process.}} \tag{7.1}$$

The water management processes considered include:

1. storage in reservoirs and
2. conveyance through canals or pipelines.

Water is considered available for beneficial use if:

1. It has been stored until it is available to be transported.
2. It has been transported through one portion of the conveyance system and is available to be transported through another portion of the conveyance system.
3. It has been transported through one portion of the conveyance system and is available to be received by the farmer.
4. It has been received by the farmer and made available for enhanced crop production (application process).

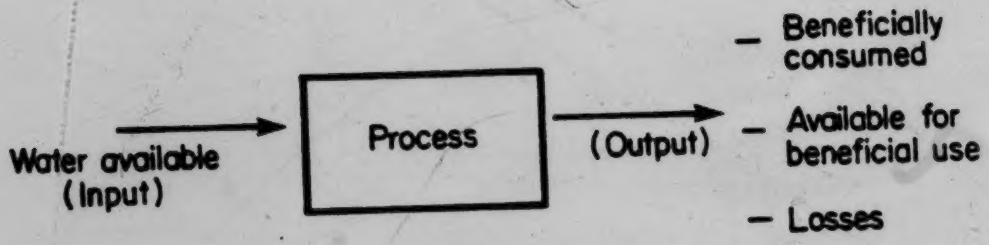

Water available
(Input)
Process
(Output)
– Beneficially consumed
– Available for beneficial use
– Losses

Figure 7.1 Schematic of water balance which may be applied to evaluate irrigation conveyance system water management processes.

Water not beneficially consumed or water not available for beneficial use or consumption at the completion of a given management process is said to be "lost".

7.4.2 Reservoir Storage Efficiency

A schematic of the water balance of a typical storage reservoir is shown in Figure 7.2. The water balance equation for a reservoir (neglecting non-irrigation water use of the reservoir) may be written as:

$$v_{u,r} + (v_{d,r} + v_{p,r}) - (v_{t,r} + v_{e,r} + v_{s,r} + v_{o,r}) - v_{bc,r} = \Delta v_{st,r} \qquad (7.2)$$

where:
$v_{u,r}$ = volume of inflow to the reservoir intended for irrigation use during time period, Δt,
$v_{d,r}$ = volume of drainage inflow to the reservoir during time period, [illegible]t,
$v_{p,r}$ = volume of precipitation inflow to the reservoir during time period, Δt,
$v_{t,r}$ = volume of transpiration outflow from reservoir during time period, Δt,
$v_{e,r}$ = volume of evaporation outflow from reservoir during time period, Δt,
$v_{s,r}$ = volume of seepage outflow from reservoir during time period, Δt,

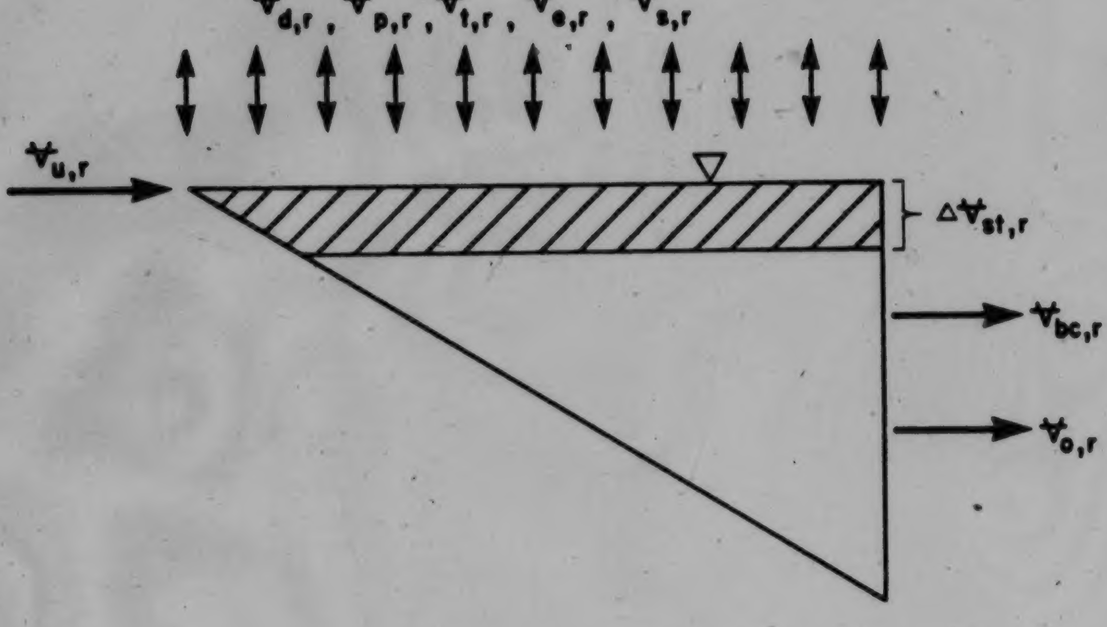

$\forall_{d,r}$, $\forall_{p,r}$, $\forall_{t,r}$, $\forall_{e,r}$, $\forall_{s,r}$
$\forall_{u,r}$
$\Delta\forall_{st,r}$
$\forall_{bc,r}$
$\forall_{o,r}$

Figure 7.2 Water balance components of a storage facility.

$v_{o,r}$ = volume of operational losses from reservoir during time period, Δt,

$v_{bc,r}$ = volume of water delivered from reservoir for irrigation purposes during time period, Δt, and

$\Delta v_{st,r}$ = change in the volume of water stored in the reservoir over time period, Δt.

Storage efficiency is defined by the relation:

$$e_r = \frac{v_{bc,r} + \Delta v_{st,r}}{v_{u,r}} \tag{7.3}$$

The storage efficiency as defined by Equation 7.3 is only marginally useful for comparing the performance of different reservoirs. An additional term related to the live storage capacity of the reservoir is required. This term, called the reservoir loss rate ratio, is defined as the water loss rate from a reservoir per unit volume of live storage at full supply level. It is defined by the relation:

$$r_L = \frac{(1 - e_r)v_{u,r}}{v_{st,r,fsl}} \tag{7.4}$$

where $v_{st,r,fsl}$ = volume of live storage at full supply level. Reservoirs with large surface area to volume ratios would likely exhibit greater loss rate ratios.

Consider the following example which compares the performance of two reservoirs.

Example

	Reservoir A	Reservoir B
$v_{u,r}$	100,000 dam^3	5,000 dam^3
$\Delta v_{st,r}$	5,000 dam^3	0
$v_{bc,r}$	80,000 dam^3	4,500 dam^3
$v_{st,r,fsl}$	100,000 dam^3	2,000 dam^3
e_r	.85	.9
r_L	.15	.25

Though reservoir A has a lower storage efficiency than B, it loses less water per unit volume of live storage than reservoir B and may be considered to perform the water storage function better than B.

7.4.3 Conveyance Efficiency

A schematic of the water balance of a typical reach of a conveyance system is shown in Figure 7.3. The water balance equation for the reach (neglecting non-irrigation water uses of the reach) may be written as:

$$v_{u,c} + L(v_{d,c} + v_{p,c}) - L(v_{e,c} + v_{t,c} + v_{s,c}) - v_{o,c} - v_{bc,c} = \Delta v_{st,c} \cdot L \qquad (7.5)$$

where: $v_{u,c}$ = volume of controlled water inflow into reach during time period, Δt,

$v_{d,c}$ = volume of drainage inflow to reach per unit length of reach during time period, Δt,

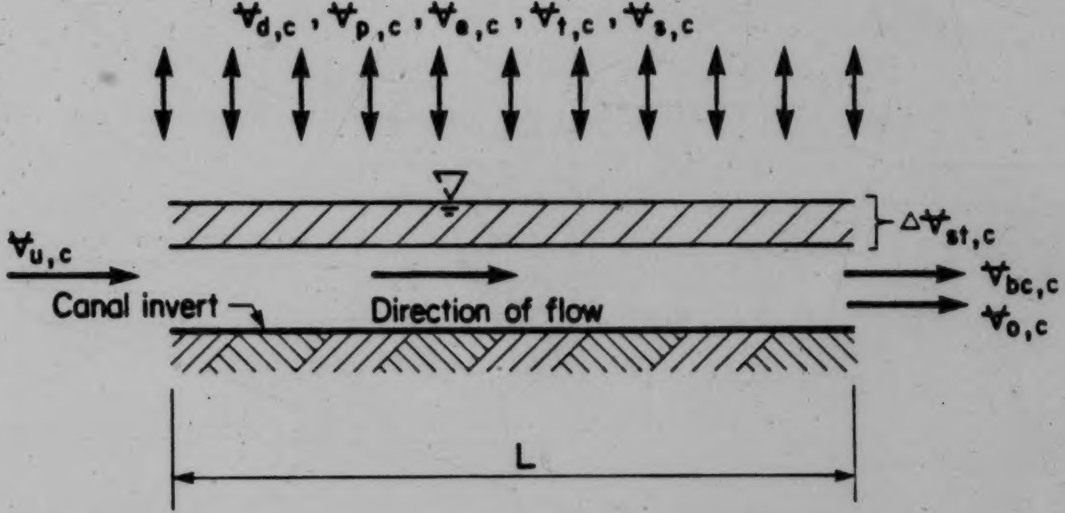

∀d,c , ∀p,c , ∀e,c , ∀t,c , ∀s,c
Δ∀st,c
∀u,c
∀bc,c
∀o,c
Canal invert
Direction of flow
L

Figure 7.3 Water balance components of a canal.

$v_{p,c}$ = volume of precipitation inflow to reach per unit length of reach during time period, Δt,

$v_{t,c}$ = volume of transpiration outflow from reach per unit length of reach during time period, Δt,

$v_{e,c}$ = volume of evaporation outflow from reach per unit length of reach during time period, Δt,

$v_{s,c}$ = volume of seepage outflow from reach per unit length of reach during time period, Δt,

$v_{o,c}$ = volume of operational losses from reach during time period, Δt,

$v_{bc,c}$ = volume of water diverted from reach for irrigation purposes during time period, Δt,

$\Delta v_{st,c}$ = change in volume of water stored per unit length of reach over time period, Δt, and

L = length of reach.

The conveyance efficiency of a reach of length, L, is defined by the relation:

$$e_c = \frac{v_{bc,c} + \Delta v_{st,c}}{v_{u,c}} \tag{7.6}$$

If the time period, Δt, is sufficiently long, $\Delta v_{st,c} \cong 0$ and Equation (7.6) may be written as

$$e_c = \frac{v_{bc,c}}{v_{u,c}} \tag{7.7}$$

Conveyance efficiency considers all factors which determine the performance of a canal or pipeline.

7.4.4 Physical Conveyance Efficiency

Physical conveyance efficiency is a measure of the relative magnitude of distributed loss rates associated with a particular canal or pipeline. It is defined by the following relation:

$$e_f = \frac{v_{u,c} - (v_{s,c} + v_{t,c} + v_{e,c})}{v_{u,c}} \qquad (7.8)$$

7.4.5 Unit Physical Conveyance Efficiency

Unit physical conveyance efficiency, Δe_f, is defined as the physical conveyance efficiency of a canal or pipeline of unit length (e.g. 1 kilometer). The use of this term permits the otherwise difficult exercise of comparing the performance of physically different canals and predicting the physical conveyance efficiency of similar canals of different length.

Distributed losses from a canal or pipeline may be related using equations of the following types:

$$\frac{dQ}{dx} = -k_1 Q \qquad (7.9)$$

where: Q is the discharge in the canal or pipeline;

x is the distance along the canal or pipeline; and,

k_1 is a proportionality constant.

or

$$\frac{dQ}{dx} = -k_2 \tag{7.10}$$

where k_2 is a constant. (See Appendix E.)

Consider Equation (7.9). If it is rearranged and integrated

$$\int_{Q_1}^{Q_2} \frac{dQ}{-k_1 Q} = \int_{X_1}^{X_2} dx \tag{7.11}$$

or

$$\ln(Q_2/Q_1) = -k_1(X_2 - X_1) \tag{7.12}$$

where X_1 and X_2 are the distances along the canal from some arbitrary starting point and Q_1 and Q_2 are the corresponding discharges at these locations. Noting that physical conveyance efficiency is defined as the ratio Q_2/Q_1 Equation (7.12) may be rearranged to yield

$$\ln(e_f) = -k_1(X_2 - X_1) \tag{7.13}$$

or $$e_f = \exp(-k_1(X_2 - X_1)) \tag{7.14}$$

The physical conveyance efficiency of a reach 1 $_{\Delta}X$ unit long may be given by the relation

$$e_{f,\Delta X} = \exp(-K_1 {}_{\Delta}X) \tag{7.15}$$

Similarly, the physical conveyance efficiency of a reach $2_{\Delta}X$ units long may be given by the relation:

$$e_{f,2_{\Delta}X} = \exp(-K_1 2_{\Delta}X) \tag{7.16}$$

and for a reach $L_{\Delta}X$ units long by the relation:

$$e_{f,L_{\Delta}X} = \exp(-K_1 L_{\Delta}X) \tag{7.17}$$

If u_f is defined as the physical conveyance efficiency of a canal 1 $_{\Delta}X$ unit long; that is

$$u_f = e_{f,\Delta X} \tag{7.18}$$

then the physical conveyance efficiency of a canal two units long is

$$e_{f,2_{\Delta}X} = \exp(-K_1 2_{\Delta}X) = u_f^2 \tag{7.19}$$

and the physical conveyance efficiency of a canal L units long is

$$e_{f,L_{\Delta}X} = \exp(-K_1 L_{\Delta}X) = u_f^L \tag{7.20}$$

or if it is recognized that u_f is defined in the same units as L,

$$e_f = u_f^L \tag{7.21}$$

Conversely,

$$u_f = e_f^{1/L} \tag{7.22}$$

The following examples illustrate the utility of the unit physical conveyance efficiency concept.

Example 1

The performance of two different canals, Reach A and Reach B, may be evaluated and compared given the necessary water balance information and canal length.

	Reach A	Reach B
v_{uc}	10,000 dam^3	5,000 dam^3
$v_{bc,c}$	7,000 dam^3	4,000 dam^3
L	10 km	3 km
e_f	.7	.8
u_f	.9650	.9283

Examination of unit physical conveyance efficiencies indicates a more efficient channel than Reach B.

Example 2

The performance of canals where unit physical conveyance efficiency is known may be predicted. If a 15 km canal was constructed that had a unit physical conveyance efficiency similar to that of Reach B in Example 1, the physical conveyance efficiency of this canal could be estimated as follows:

$$e_f = (0.9283)^{15}$$
$$= 0.3276$$

Example 3

The use of the unit physical conveyance efficiency concept can also be used to demonstrate the variable nature of the physical conveyance efficiency of conveyance subsystems.

A simple conveyance subsystem is illustrated in Figure 7.4. The unit physical conveyance efficiency of all reaches is 0.90. Quaternary off-takes are located at B, C, and D. The primary distributary off-take is located at A. Assuming no operational spill or significant lag-time in the water deliveries and farm water demands as indicated, the required diversions through the primary distributary off-take and the physical conveyance efficiency for each day may be estimated.

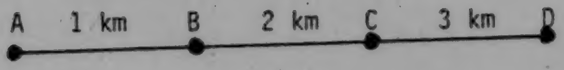

A
1 km
B
2 km
C
3 km
D

Figure 7.4 Distributary canal discussed in Example 3.

Farm Water Demand l/s

Day	B	C	D	Required Diversion l/s	e_f
1	100	50	50	273.8	0.73
2	50	0	100	243.7	0.62

The required diversions may be calculated using the following relation:

$$q_A = \frac{q_B}{(u_f)^1} + \frac{q_C}{(u_f)^3} + \frac{q_D}{(u_f)^6}$$

where q_B, q_C, and q_D correspond to the farm water demands at quaternary off-takes B, C, and D respectively and $u_f = 0.90$. The physical conveyance efficiency for the system is calculated using the relation:

$$e_f = \frac{q_B + q_C + q_D}{q_A}$$

Note that the physical conveyance efficiency considering both days is 0.68.

Note that the unit physical conveyance efficiency concept is only valid if the rate of distributed losses is described by Equation 7.9. Consider Equation (7.10) which is restated here for convenience

$$\frac{dQ}{dx} = -K_2 \tag{7.10}$$

If this equation is rearranged and integrated

$$\int_{Q_1}^{Q_2} dQ = -K_2 \int_{X_1}^{X_2} dx \qquad (7.23)$$

we obtain

$$Q_2 - Q_1 = -K_2(X_2 - X_1) \qquad (7.24)$$

Rearranging and dividing by Q_1 the following expression for physical conveyance efficiency is obtained:

$$e_f = 1 - \frac{K_2(X_2 - X_1)}{Q_1} \qquad (7.25)$$

Note that in this case, unlike in Equation (7.21), the physical conveyance efficiency of a reach one unit long increases with increasing discharge.

7.4.6 Operational Efficiency

Operational efficiency is a measure of the ability of the canal or pipeline operations to minimize distributed water losses and spillage while completely satisfying water demands. Operational efficiency is defined by the relation:

$$e_o = \frac{v_{bc,c} + (v_s + v_e + v_t)_{min}}{v_{u,c}} \qquad (7.26)$$

where $(v_s + v_e + v_t)_{min}$ is the minimum distributed loss incurred while delivering the volume of water, $v_{bc,c}$.

7.4.7 Maximum Conveyance Efficiency

The maximum attainable conveyance efficiency, $e_{c,max}$, will occur when the volume of spillage equals zero and the volume of distributed losses incurred while making the water delivery(ies) is at a minimum. Maximum conveyance efficiency is therefore defined by the relation:

$$e_{c,max} = \frac{v_{bc,c}}{(v_{u,c})_{min}} = \frac{v_{bc,c}}{v_{bc,c} + (v_s + v_t + v_e)_{min}} \qquad (7.27)$$

where the volume of inflow to the canal or pipeline reach is the minimum required to make the water deliveries.

7.4.8 Example Demonstrating the Use of Efficiency Terms

Table 7.1 which compares the expected efficiencies associated with unlined canals, lined canals, and closed pipeline systems.

TABLE 7.1

Comparison of Efficiencies of Unlined Canals, Lined Canals, and Closed Pipelines

	e_c	e_f	u_f	e_o	$e_{c,max}$
unlined canal	<1	<1	<1	≤ 1	<1
lined canal	≤ 1	$=1$	$=1$	≤ 1	$=1$
closed pipeline	$=1$	$=1$	$=1$	$=1$	$=1$

7.5 Discussion

All of the parameters with which the performance of irrigation conveyance systems may be evaluated are quantifiable. However, the field observations required are apparently difficult or expensive to obtain Manz (1984(b)) and Appendix K. This difficulty has prevented the development of conveyance system performance evaluation techniques, such as the ones introduced in Section 7.4, because of their immediate designation as being impractical. However, with the availability of extensive and inexpensive computational capability, this situation need not continue.

As will be demonstrated in Chapter 8, computer models can be developed which are capable of simulating conveyance system operations with sufficient detail to specify and/or control the parameters with which the performance of irrigation conveyance systems may be evaluated. The utility of such models is significant in that no other reasonable options exist by which the impact of the various physical and operational characteristics of the irrigation conveyance system and its environment can be examined.

CHAPTER 8

THE IRRIGATION CONVEYANCE SYSTEM SIMULATION MODEL

8.1 General Description

The development of the Irrigation Conveyance System Simulation Model (ICSS) required a clear description of the simulation problem. A very important part of the problem description is a clear statement of the scope of the problem the model is intending to address. This was provided by the terminology and classification systems used to identify the farm water demand the irrigation conveyance systems are intended to satisfy, the operational and hydraulic characteristics of the individual components which comprise irrigation conveyance systems, the manner with which the various components could be assembled to form irrigation conveyance systems and subsystems and, lastly, the techniques available to operate the various conveyance systems and subsystems. Using the terminology and classification system developed and presented in Chapters 3, 4, 5, and 6, it is possible to state the capability of the ICSS Model in a manner meaningful not only to engineers who would apply the model but also to "modellers" interested in extending the model. (It is also possible to clearly state that part of the problem scope the model does not accommodate). The second important part of the problem description is a statement of the model's "purpose". In the case of the ICSS Model, its purpose is primarily to aid evaluation of the operation of irrigation conveyance systems as described in Chapter 7. However, it is capable of being used for a multitude of other evaluation and design purposes. The model scope and purpose together defined the particular operational, hydraulic and hydrologic phenomena the model is required to simulate. It was discovered that for the model to be general, a great variety of operational, hydraulic and hydrologic phenomena would have to be incorporated. This was not considered practical, at least not at this time. Instead, it was considered practical to provide the opportunity to simulate, within the same model structure, as great a variety of operational, hydraulic and hydrologic phenomena the model user may wish to include. The particular algorithms chosen for inclusion in the version of the model presented were selected on the basis of their utility in simulating the phenomena of interest, their precision and their ease of incorporation into the model. (The algorithms used to simulate the hydraulic and hydrologic phenomena incorporated in the model and the reasons for their selection are described in Appendices A to G.) It is recognized that many other algorithms describing exactly the same phenomena could have been chosen and new algorithms are constantly being developed. Hence to maintain the utility and relevance of the ICSS Model it was decided that provision should also be made within the model structure for replacement of particular algorithms, or the provision of a "menu" of algorithms. The version of the ICSS Model presented demonstrates the utility of the various techniques introduced to describe and evaluate irrigation conveyance systems and the potential use these descriptive techniques have for improving the irrigation conveyance system function - the expeditious delivery of water from its source to the farmer-water consumer.

The particular version of the model (a listing of the program is provided in Appendix H) simulates the hydraulics of flow in unlined or lined irrigation canals. The individual canals may contain any number of distinct prismatic reaches of various cross-sectional shape and roughness. Each reach is bounded by one or several positive and/or upstream hydraulically controlled, manually operated or fixed hydraulic structures. A limited number of types and combinations of hydraulic structures are included in the model and may be selected at the user's option. (As many types and variations of hydraulic structures as the user may require can be incorporated at a later time). Similarly, though provision is made to consider seepage, precipitation, drainage and evaporation or any other distributed losses or gains, only one method of seepage calculation is included. Resistance to flow in the canals, as described by Manning's roughness, may be constant or provision is made for the inclusion of subroutines which permit it to vary as the user requires. It is important to note that each reach of the canal considered may have entirely different characteristics. This version of the model is intended to evaluate irrigation canals operated using upstream control. The model is written in Fortran 5 and is executed interactively on a Control Data Corporation Cyber 175 computer system.

Output from the ICSS Model is controlled by the user and may include the following information:

1. a restatement of input data;

2. the initial water surface profiles, velocities, and discharges throughout each reach;
3. the water surface profiles and velocities, as they vary with time, for each reach;
4. the water depth and discharge, as they vary with time, at the inlet and outlet of each reach;
5. an instantaneous and cumulative statement of the water balance for each reach;
6. a record of all operations performed during a particular run; and,
7. a record of hydraulic structure status, flow condition, and water inflow and outflow at the inlet and outlet of each reach.

Additional output options are provided for but not implemented at this time since sufficient output is provided to evaluate the operations of any irrigation canal considered.

Computer programs used to set up input data files which control an entire run and input data files for each reach are listed in Appendices I and J respectively.

8.2 Relationship of ICSS to Other Computer Models of Irrigation Conveyance Systems

To the writer's knowledge computer models of irrigation conveyance systems with capabilities similar to ICSS do not exist. The Unsteady

Flow Model (USM), and Gate Stroking Model (GSM) developed by the United States Bureau of Reclamation are intended for the development of techniques for the stable operation of irrigation canals using automatically controlled hydraulic structures Falvey and Luning (1979) and Bulyaski and Serfozo (1979). These models were recently employed by Burt (1982) to develop an algorithm for automatic downstream control of an irrigation canal. Both the USM and GSM are intended to examine hydraulic transients resulting from hydraulic structure operation. (This information is provided by ICSS as well.) The design of the California Aquaduct and the development of its operational procedures employed extensive computer simulation Frederiksen (1969). The computer model used was developed by Amorocho and Strelkoff (1965) for the California Department of Water Resources. This model is concerned with flow control and the minimization of hydraulic transients, that is, the development and refinement of combined automatic and manual operational procedures. It was recently employed by Zimbelman and Bedworth (1983) to develop an algorithm for automatic downstream operation of irrigation canals. Similar to the computer model used to simulate the operation of the California Aquaduct, the computer simulation of the operation of the Canal de Provence in France in 1963 and 1964, as described by Cunge and Woolhiser (1975) is also exclusively concerned with the control and stability of flows and water supply in the complex water delivery system. Cunge et al. (1980) discuss the use of computer modelling of irrigation canals with automatic control devices to perform control and stability analysis. (ICSS may be modified to include automatically operated hydraulic structures as well as the manually operated structures now being considered.) Ploss et al. (1979) discuss the use of a computer model developed by the United States Bureau of Reclamation to perform water delivery scheduling (not to be confused with on-farm irrigation scheduling) within irrigation conveyance systems. This model does not simulate the hydraulic characteristics of the conveyance system but is provided the necessary information as input (e.g. time-of-travel) which may be determined from field observations or estimated using a model such as ICSS. The model described by Ploss et al. (1979) is directed at conveyance systems which employ operations that are upstream controlled and which satisfy collaboratively controlled farm water demands. Unlike the ICSS Model the aforementioned models neither attempt to nor are intended for relating conveyance system performance as described in Chapter 7 to the interaction between the physical characteristics of irrigation conveyance systems and the manner in which they are operated. None of the aforementioned models consider distributed water losses or gains in the manner of the ICSS Model or permit interactive execution.

8.3 Detailed Description of the Irrigation Conveyance System Simulation Model

8.3.1 Overview

The ICSS Model was designed to provide the model operator (user) with maximum flexibility in the inclusion, selection, and application of algorithms which may be used to simulate the physical, environmental, and operational characteristics of irrigation conveyance systems. The

ICSS Model is modular in structure and attempts to isolate each characteristic being simulated so that algorithms developed to simulate each characteristic may be included, replaced, or selected without requiring the alteration of the logic in the remainder of the model. Flexibility of application is further increased through the interactive execution feature.

The ICSS Model was developed as a research tool which could continue to evolve while serving many useful functions at each stage of its development (e.g. such as the performance evaluation of irrigation conveyance systems). The version of the ICSS Model presented must be considered little more than the basic structure on which the model can grow.

8.3.2 Structure of Model

8.3.2.1 General

A flow chart of the ICSS Model is shown in Figure 8.1. There are four primary segments; initialization, operations, hydrological calculations, and unsteady flow calculations. The initialization segment assigns values to the variables which control individual simulations, which describe the physical nature of the canal being studied and which are required for the satisfactory initiation of the various numerical analyses implemented within the model. The operations segment facilitates the interactive review and operation of those hydraulic structures which are operable and the review of those hydraulic structures which are fixed. The hydrological calculation

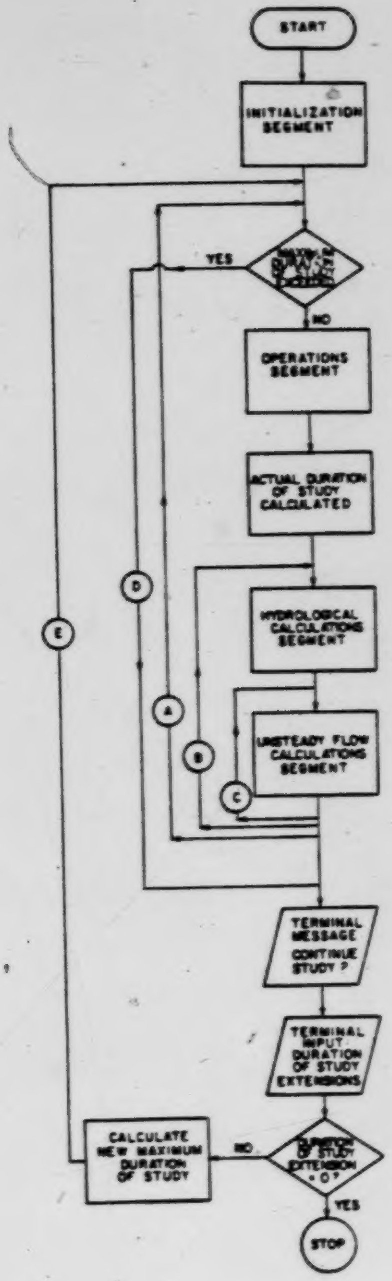

START
INITIALIZATION SEGMENT
YES
NO
OPERATIONS SEGMENT
ACTUAL DURATION OF STUDY CALCULATED
HYDROLOGICAL CALCULATIONS SEGMENT
UNSTEADY FLOW CALCULATIONS SEGMENT
A
B
C
D
E
TERMINAL MESSAGE CONTINUE STUDY ?
TERMINAL INPUT DURATION OF STUDY EXTENSIONS
DURATION OF STUDY EXTENSION = 0 ?
NO
CALCULATE NEW MAXIMUM DURATION OF STUDY
YES
STOP

Figure 8.1 Flow Chart of ICSS Model.

segment may update the 'hydraulic roughness', seepage, evaporation, precipitation, and any other distributed lateral inflow or outflow characteristics of each reach. The unsteady flow calculation segment simulates the transient flow condition in each reach of the canal being studied.

The segments are related as follows:

1. Once the initialization segment is performed, the dynamic portion of the model becomes operative. The first process performed is a test to determine if the maximum duration of the study, specified in the run control data, has been exceeded. If it hasn't been exceeded, the model operator is given the opportunity to review and/or operate all operable hydraulic structures and review all fixed hydraulic structures. This is performed interactively on the computer terminal with information, messages, and requests being displayed on the terminal to which the model operator will respond. As well as controlling the operation of the hydraulic structures, the operator will specify when he wants the opportunity to operate the structures again. The actual duration of the study is then established.
2. The hydraulic roughness, seepage, evaporation, precipitation, and other distributed lateral inflow or outflow processes are evaluated. If the simulation is about to start, these values will remain unchanged as the simulation enters the unsteady flow calculation segment. If hydraulic structures have been operated and the unsteady flow calculation segment has also

functioned, those hydrological characteristics which are a function of flow conditions will have changed. It will be noted that the frequency of hydrological calculations is controlled by loop B and a time interval, called the "hydrological time increment", which has previously been specified by the model operator.

3. The unsteady flow calculation segment will route the water through the various reaches of the canal. If no hydraulic structures have been operated and if the hydrological conditions have not been altered, the flow conditions in the canal remain unchanged. If hydraulic structures have been operated or if the hydrological characteristics of the canal have changed, the unsteady flow calculation segment will determine the flow condition in previously specified time increments, called the "hydraulic routing time intervals" in the cycle indicated by loop C. Once loop C is satisfied, the control returns to loop B and then to loop A.
4. If the maximum duration of study has been exceeded the model operator is informed on the terminal and given the opportunity to increase the maximum duration and continue the study. If he does not wish to continue, the simulation is stopped.

8.3.2.2 Initialization

8.3.2.2.1 Function

The following functions are performed in the initialization segment:

1. initialization of run control variables;
2. initialization of reach specific variables;
3. initialization of cumulative water balance terms;
4. initialization of water depth and velocity at each node of each reach;
5. initialization of coefficients, necessary to commence unsteady flow routing, which are a function of initial flow conditions at the upstream and downstream boundary of each reach;
6. initialization of duration of simulation variable; and,
7. writing to file of the initial water balance, reach inlet and outlet flow data, water delivery statistics, check/weir statistics, downstream boundary statistics, upstream boundary statistics and the depth and velocity at each node in the reach for all reaches.

A flow chart illustrating the sequence with which the different functions are performed is shown in Figure 8.2.

8.3.2.2.2 Initialization of Run Control Variables

All data required to initialize the run control variables are read by the main program from a data file named SIMDAT 2 created using the SIMDAT program listed in Appendix I.

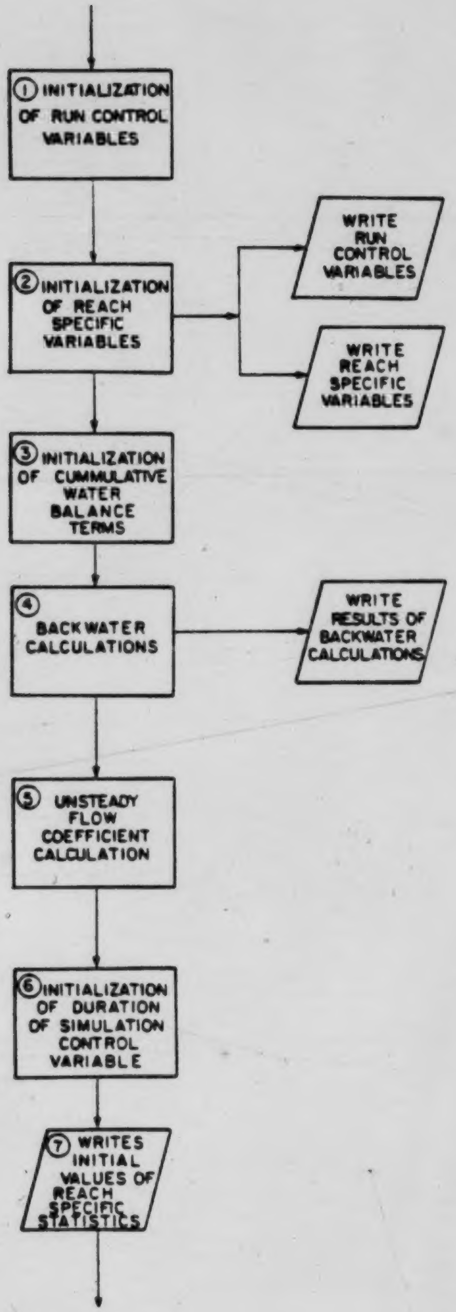

① INITIALIZATION OF RUN CONTROL VARIABLES
② INITIALIZATION OF REACH SPECIFIC VARIABLES
WRITE RUN CONTROL VARIABLES
WRITE REACH SPECIFIC VARIABLES
③ INITIALIZATION OF CUMMULATIVE WATER BALANCE TERMS
④ BACKWATER CALCULATIONS
WRITE RESULTS OF BACKWATER CALCULATIONS
⑤ UNSTEADY FLOW COEFFICIENT CALCULATION
⑥ INITIALIZATION OF DURATION OF SIMULATION CONTROL VARIABLE
⑦ WRITES INITIAL VALUES OF REACH SPECIFIC STATISTICS

Figure 8.2 Flow Chart of Initialization Segment.

The data which is read includes:

- duration of simulation,
- time of day simulation starts,
- hydraulic routing time increments,
- hydrological time increments,
- total number of reaches in the canal,
- weighting factor used in unsteady flow calculations,
- one minus the weighting factor,
- maximum value of residual used to determine convergence in the unsteady flow calculations, and
- the gravitational constant.

8.3.2.2.3 Initialization of Reach Specific Variables

All data required to initialize the reach specific variables is read by a subroutine named DATAM from a file named RCHDAT 7 created using the program RCHDAT. (A listing of the program RCHDAT may be found in Appendix J.) Several variables, whose values are not obtained directly from the file RCHDAT 7, are calculated within the subroutine DATAM itself. The variables which are either read by DATAM or calculated by it include the following:

1. Reach indices for each reach. These are indicators of the operability of the upstream or downstream boundary of each reach and of the variability of the roughness or hydrological characteristics of each reach. If the index equals one, the boundary is not operable or the characteristic is not

variable. If the index equals two, the boundary is operable or the characteristic is variable. Values are contained in the matrix containing the reach indices, ICHID (number of reaches, index), which currently provides for a maximum of ten reaches and ten indices which are defined as follows:

- one, reach number,
- two, roughness,
- three, total distributed lateral inflow/outflow,
- four, seepage,
- five, precipitation,
- six, evaporation,
- seven, other distributed lateral inflow/outflow,
- eight, upstream boundary,
- nine, downstream boundary, and
- ten, is not used at this time.

2. Reach parameters for each reach, which are contained in the matrix RCHPR (number of reaches, parameter), and which currently provide for a maximum of ten reaches and five parameters provide the following information:
    - one, length of reach,
    - two, total number of equally spaced nodes in reach at which depth and velocity are determined,
    - three, distance between nodes,
    - four, ratio of hydraulic routing time increment and distance between nodes, and
    - five, the full supply level of the reach.

3. Elevation of canal bottom at each node in each reach contained in matrix, ELEV (number of reach, node), which currently provides for a maximum of ten reaches and twenty-one nodes.
4. Channel geometry coefficients for each reach, which are contained in matrix GEOM (number of reach, coefficient) and which currently provides for a maximum of ten reaches and twenty-one coefficients. Each reach is assumed to be prismatic and to possess a cross-section whose area, wetted perimeter and top width vary with canal depth according to a third order polynomial. The first four coefficients of GEOM are the constant and leading coefficients of canal depth to the first, second, and third power contained in the equation defining canal cross-sectional area; the next four coefficients pertain to the equation describing top width and the four coefficients following that pertain to the equation describing wetted perimeter. Differentiation of the preceding three equations, with respect to canal depth (prior to programming) produce second order polynomials whose coefficients are also contained in GEOM in three additional groups of three coefficients pertaining to area, top width, and wetted perimeter respectively.

It may be noted at this time that the third order polynomial fits rectangular and trapezoidal shaped canal cross-sections exactly and will fit many irregularly shaped canal cross-sections as well. Canal area, top width, wetted perimeter, and their derivatives with respect to depth are made available throughout the program using function subroutines

AREA, TOPW, WETP, DAREA, DTOPW, and DWETP. If canal cross-sections of a more complex shape must be considered, higher order polynomials may be used instead.

5. Roughness parameters for each reach are contained in matrix ROUGH (number of reach, parameter) which currently provides for a maximum of ten reaches and twenty parameters. The parameters provide the following information:
    - one, total number of parameters used to describe the roughness characteristics of reach,
    - two, estimate of initial roughness of reach,
    - three, variability of roughness, (one if variable; two if constant),
    - four, type code of subroutine used to describe hydraulic roughness if it is variable, and,
    - five to twenty, parameters which may be used to describe the specific type of roughness variability experienced by reach.
6. Initial estimate of total distributed lateral inflow or outflow to or from reach contained in matrix QLAT (number of reach) which provides for a maximum of ten reaches.
7. Seepage parameters for each reach contained in matrix OSEEP (number of reach, paramater), which currently provides for a maximum of ten reaches and twenty parameters. The parameters provide the following information:
    - one, total number of parameters used to describe the seepage characteristics of the reach,

- two, estimate of initial seepage from reach,
- three, variability of seepage, (equal to one if variable; two if constant),
- four, type code of subroutine used to describe seepage if it is variable, and
- five to twenty, parameters which are specific to the type of seepage variability experienced by reach.

8. Precipitation parameters for each reach contained in matrix QPREC (number of reach, parameter), which currently provides for a maximum of ten reaches and twenty parameters. The parameters provide the following information:
    - one, total number of parameters used to describe the precipitation characteristics of reach,
    - two, initial precipitation into reach,
    - three, variability of precipitation, (equal to one if variable, two if constant),
    - four, type code of subroutine used to describe precipitation if it is variable, and
    - five to twenty, parameters which are specific to the type of precipitation variability experienced by reach.
9. Evaporation parameters for each reach contained in matrix QEVAP (number of reach, parameter), which currently provides for a maximum of ten reaches and twenty parameters. The parameters provide the following information:

- one, total number of parameters used to describe the evaporation characteristics of the reach;
- two, estimate of initial evaporation from reach;
- three, variability of evaporation (equal to one if variable, two if constant); and,
- four, type code of subroutine used to describe evaporation if it is variable, and
- five to twenty, parameters which are specific to type of evaporation variability experienced by reach.

10. Other distributed lateral inflow or outflow parameters for each reach contained in matrix Q0DST (number of reach, parameter) which currently provides for a maximum of ten reaches and twenty parameters. The parameters provide, in sequence, the following information:
    - one, total number of parameters used to describe the particular type of distributed lateral inflow or outflow being considered;
    - two, estimate of initial inflow or outflow to or from reach,
    - three, variability of the other distributed lateral inflow or outflow (equal to one if variable, two if constant),
    - four, type code of subroutine used to describe the other distributed lateral inflow or outflow if variable, and,

- five to twenty, parameters which are specific to the type of other distributed lateral inflow or outflow variability experienced by reach.

11. Upstream boundary parameters for each reach are contained in matrix UBCPR (number of reach, parameter) which currently provides for a maximum of ten reaches and fifty parameters. The parameters provide, in sequence, the following information:

    - one, total number of parameters used to describe the particular type of upstream boundary condition considered,
    - two, the type of upstream boundary condition considered (provided as a type code),
    - three, the operability of structure, (a value of one indicates a fixed structure, two indicates an operable structure),
    - four, total rate of flow at the reach inlet,
    - five, depth of flow at the reach inlet,
    - six, unsteady flow coefficient, $\partial G_0/\partial y_1{}^{j+1}$,
    - seven, unsteady flow coefficient, $\partial G_0/\partial V_1{}^{j+1}$,
    - eight, unsteady flow coefficient, $-r^k_{2,0}$,

    (See Appendix A for explanation of parameters six, seven and eight.)

    - nine, rate of inflow from other sources,
    - ten, rate of inflow from upstream reach, and

- eleven to fifty are specific to type of upstream boundary.

12. Downstream boundary parameters for each reach are contained in matrix DBCPR (number of reach, parameter) which currently provides for a maximum of ten reaches and sixty parameters. The parameters provide, in sequence, the following information:

    - one, total number of parameters used to describe the particular type of downstream boundary condition considered,
    - two, the type of downstream boundary condition considered, (provided as a type code),
    - three, the operability of the structure, (a value of one indicates a fixed structure, two indicates an operable structure); and,
    - four, total rate of flow at the end of the reach,
    - five, depth of flow at the end of the reach,
    - six, unsteady flow coefficient, $\partial F^{j+1}/\partial y_N^{j+1}$,
    - seven, unsteady flow coefficient, $\partial F_N/\partial V_N^{j+1}$,
    - eight, unsteady flow coefficient, $-R_{1,N}^{k}$,

    (See Appendix A for explanation of parameters six, seven and eight.)

    - nine, total useful water delivery,
    - ten, actual flow to the downstream reach,
    - eleven to seventeen, specific to type of downstream boundary,

- eighteen, observed flow to downstream reach,
- nineteen, calculated minimum flow to downstream reach,
- twenty, simulation flow to downstream reach,
- twenty-one to twenty-five, specific to type of downstream boundary,
- twenty-six, farm water demand,
- twenty-seven, actual water delivery to farm,
- twenty-eight and twenty-nine, upper and lower tolerance limits for errors in the actual farm water delivery,
- thirty to thirty-three, specific to type of downstream boundary,
- thirty-four, downstream water requirements,
- thirty-five and thirty-six, upper and lower tolerance limits to errors in the actual flow intended to satisfy downstream water requirements, and
- thirty-seven, useful flow to downstream.

13. Report control indices for each reach contained in matrix IREP (number of reach, indices) which currently provides for a maximum of ten reaches and twelve indices. If the index equals one, the information is written to a file which may be accessed at a later time. If the index equals two the information is not written to file. The output from the program is controlled as follows:

- index number one controls the writing of the initial values of all run control variables and all reach specific variables,
- index number two controls the writing of the results of backwater calculations,
- index number three controls the writing of check status as it varies with time,
- index number four controls the writing of water delivery number one status as it varies with time,
- index number five controls the writing of water delivery number two status as it varies with time,

(Each reach may have two water delivery points.)

- index number six controls the writing of water depth and velocity at each node as they vary with time,
- index number seven controls the writing of depth and discharge at the inlet and outlet as they vary with time,
- index number eight controls the writing of the operations log,
- index number nine controls the writing of the instantaneous and cumulative water balance as it varies with time,
- index number ten is presently inactive,
- index number eleven controls the writing of the upstream boundary parameters as they vary with time, and

- index number twelve controls the writing of the downstream boundary parameters as they vary with time.

Depending on the value of report control index number one, both the run control variables and the reach specific variables may be written to a file named DATAM1.

8.3.2.2.4 Backwater Calculations

Backwater calculations are performed by a subroutine called BACKW. A detailed flowchart of BACKW is shown in Figure 8.3. The calculations proceed as follows:

1. Backwater of last reach (most downstream) is calculated using initial estimates of total lateral inflow or outflow from or to the reach and initial estimates of the hydraulic roughness.

   The equations being solved are

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{2Q}{gA^2}\, p}{1 - F^2} \tag{8.1}$$

and

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{Q}{gA^2}\, i}{1 - F^2} \tag{8.2}$$

where y = water depth;

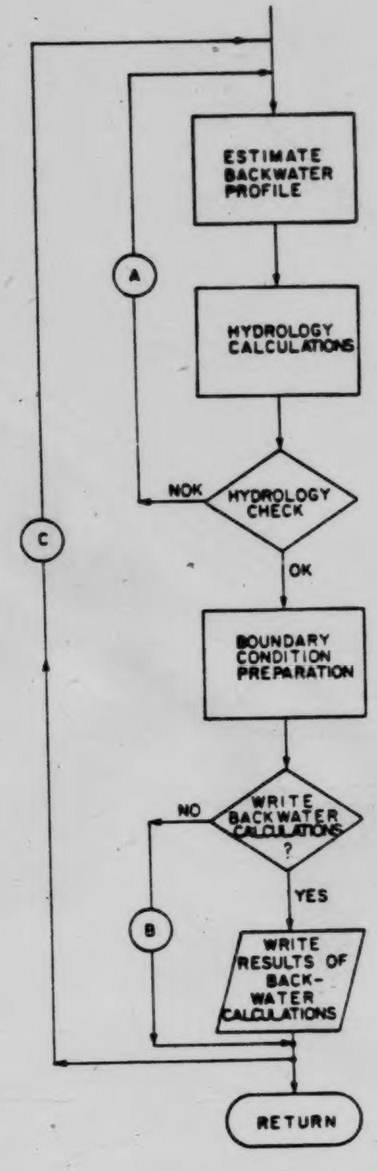

ESTIMATE BACKWATER PROFILE
A
HYDROLOGY CALCULATIONS
NOK
HYDROLOGY CHECK
C
OK
BOUNDARY CONDITION PREPARATION
NO
WRITE BACKWATER CALCULATIONS ?
YES
B
WRITE RESULTS OF BACK-WATER CALCULATIONS
RETURN

Figure 8.3 Detail flowchart of backwater calculation subroutine, BACKW.

x = distance along channel;

A = cross-sectional area;

g = acceleration due to gravity;

Q = discharge;

S_o = slope of channel bottom;

S_f = slope of total energy line;

p = rate of distributed lateral inflow;

i = rate of distributed lateral outflow; and,

F = Froude Number.

Friction slope is calculated using the Chezy-Manning Equation,

$$S_f = \frac{n^2 V^2}{R^{4/3}} \qquad (8.3)$$

where n = hydraulic roughness;

V = average velocity of flow; and,

R = hydraulic radius.

A detailed description of the numerical techniques used may be found in Appendix B.

2. Total lateral inflow or outflow from the reach and/or the hydraulic roughness are calculated. If these values are a function of reach flow conditions, values other than those estimated will likely be generated.
3. The new values of total lateral inflow or outflow and/or the hydraulic roughness will be compared to the previous values.

If there is a significant difference, Steps 1 and 2 will be repeated. If there is no significant difference, the backwater profile is assumed to be correct.

4. The upstream and downstream boundary parameters for a reach for which a backwater profile had just been calculated and the downstream boundary parameters for the reach immediately upstream are corrected.
5. The backwater profile for the reach may be written in a file named BACKW1 depending on the value assigned report control index number two.
6. Steps 1 through 5 are repeated for each reach.

8.3.2.2.5 Hydrological and Hydraulic Roughness Calculations

The value of the hydraulic roughness and total distributed lateral inflow or outflow of each reach is controlled by the subroutine named HYDROL. A detailed flowchart of HYDROL is shown in Figure 8.4. HYDROL performs as follows:

1. Reach index number two is examined to determine if the hydraulic roughness for the reach is variable. If it isn't variable, the call to subroutine ROUGH, where the roughness calculation selection procedure is performed, is skipped. If the hydraulic roughness is variable, the roughness calculation selection procedure using roughness parameter number four, will direct calculations to appropriate subroutine ROUGH1, ROUGH2, etc. (Recall that the version of ICSS presented does not contain any subroutines for recalculating hydraulic

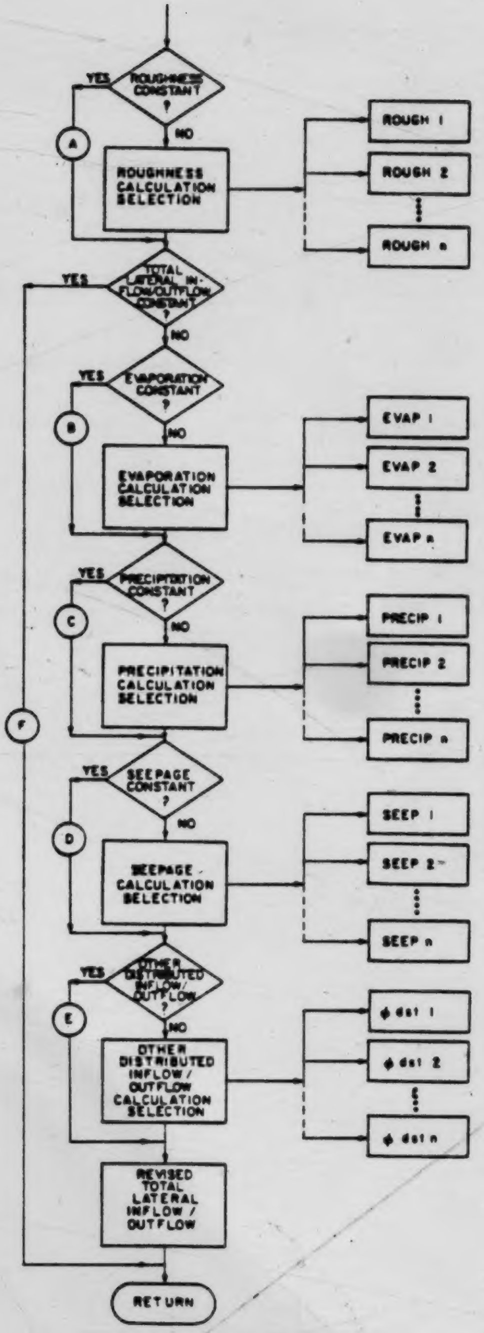

ROUGHNESS CONSTANT ?
YES
NO
A
ROUGHNESS CALCULATION SELECTION
ROUGH 1
ROUGH 2
ROUGH n
TOTAL LATERAL IN-FLOW/OUTFLOW CONSTANT ?
YES
NO
EVAPORATION CONSTANT ?
YES
NO
B
EVAPORATION CALCULATION SELECTION
EVAP 1
EVAP 2
EVAP n
PRECIPITATION CONSTANT ?
YES
NO
C
PRECIPITATION CALCULATION SELECTION
PRECIP 1
PRECIP 2
PRECIP n
F
SEEPAGE CONSTANT ?
YES
NO
D
SEEPAGE CALCULATION SELECTION
SEEP 1
SEEP 2
SEEP n
OTHER DISTRIBUTED INFLOW/OUTFLOW ?
YES
NO
E
OTHER DISTRIBUTED INFLOW/OUTFLOW CALCULATION SELECTION
φ dst 1
φ dst 2
φ dst n
REVISED TOTAL LATERAL INFLOW / OUTFLOW
RETURN

Figure 8.4 Detail flowchart of hydrology calculation subroutine, HYDROL.

roughness and that this feature is provided for in the program if required.)

2. Reach index number three is examined to determine if the total distributed lateral inflow and outflow is constant. If it is constant control is returned to the calling subroutine. If it is not constant, evaporation, precipitation, seepage and other distributed lateral inflow or outflow are examined to determine how total distributed lateral inflow or outflow for the reach has changed.

3. Reach index number six is examined to determine if evaporation is variable. If it is not variable, the call to subroutine EVAP, where the evaporation calculation selection procedure is performed is skipped. If it is variable, the evaporation calculation selection procedure, using evaporation parameter number four, will direct calculations to appropriate subroutine, EVAP1, EVAP2, etc. (Recall that the version of ICSS presented does not contain any subroutines for recalculating evaporation losses and that this feature is provided for in the program if required. A discussion of evaporation losses from canals may be found in Appendix F.)

4. Research Index number five is examined to determine if precipitation inflow to the reach is variable. If it is not variable, the call to subroutine PRECIP where the precipitation calculation selection procedure is performed, is skipped. If it is variable, the precipitation calculation selection procedure, using precipitation parameter number

four, will direct calculation to appropriate subroutine, PRECIP1, PRECIP2, etc. (Recall that the version of ICSS presented does not contain any subroutines for recalculating precipitation and that this feature is provided for in the program if required. A discussion of precipitation inflow to canals may be found in Appendix G.)

5. Reach index number four is examined to determine if seepage from the canal is variable. If it is not variable, the call to subroutine SEEP, where the seepage calculation selection procedure is performed, is skipped. If it is variable, the seepage calculation selection procedure, using seepage parameter number four, will direct calculations to the appropriate subroutine, SEEP1, SEEP2, etc. The version of ICSS presented contains only one procedure after Moritz for calculating seepage. (The subroutine SEEP does not perform the calculation selection procedure, since it is unnecessary; but simply proceeds to calculate QSEEP directly. The selection feature may be activated if required.) A discussion of the seepage losses from irrigation canals may be found in Appendix E.
6. Reach index number seven is examined to determine if other forms of distributed lateral inflow or outflow to or from the canal are variable. (Any other form of distributed lateral inflow which may be considered is included in drainage inflow from adjacent land areas. Another form of distributed lateral outflow which may be considered is a syphon water supply to

border dyke irrigation operations.) If the reach index indicates that other distributed lateral inflow is not variable, the calculation selection procedure, performed by subroutine ØDST, is skipped. If it is variable, the calculation selection procedure, using other distributed lateral inflow/outflow parameter number four, will direct calculation to the appropriate subroutine, ØDST1, ØDST2, etc. (The version of ICSS presented does not contain any subroutines for recalculating other distributed lateral inflow or outflow but this feature is provided for in the program if required.)

7. A revised total distributed lateral inflow or outflow for the reach is calculated.

The ICSS Model contains a significant restriction regarding the manner with which hydraulic roughness and the hydrological processes are simulated. The decision was made to assume that distributed lateral inflow and outflow, throughout an entire reach, would not change throughout the length of the reach. This simplification can be removed from the steady-state flow condition calculations performed in the initialization segment without adding significantly to computational requirements. Unfortunately, the same is not true in the unsteady flow calculation segment. Here a reformulation of the entire solution routine would be required and computational requirements would be expected to increase by several orders of magnitude. For reasons of economy and the uncertainty that significant benefits would accrue from the more detailed approach, the problem was reserved for future consideration when computational costs, at least, would be substantially lower than they are at present. It should be noted that the model operator is provided with the opportunity to not only apply one of the several different models of the hydrological and hydraulic roughness characteristics to different reaches of the same canal but also to change the specific nature of the same model as he sees fit.

8.3.2.2.6 Boundary Condition Preparation

The parameters describing the upstream and downstream boundary of the reach, for which the backwater calculation has just been performed, are updated using the subroutine BWPREP. A detailed flowchart of BWPREP is shown in Figure 8.5. It performs as follows;

1. Sets boundary code equal to four.
2. Calls subroutine UBC which in turn selects the control subroutine which describes the upstream boundary for the reach, UBC01, UBC02, etc., based on the second parameter of matrix UBCPR.
3. The control subroutine calls a subroutine, UBC01D, UBC02D, etc., if the boundary code equals four. These subroutines will then perform the necessary upstream boundary parameter update for the reach the backwater calculations have been performed and for the downstream boundary condition of the reach immediately upstream. The upstream boundary parameters normally updated are total discharge and depth. The downstream boundary parameters normally updated are the

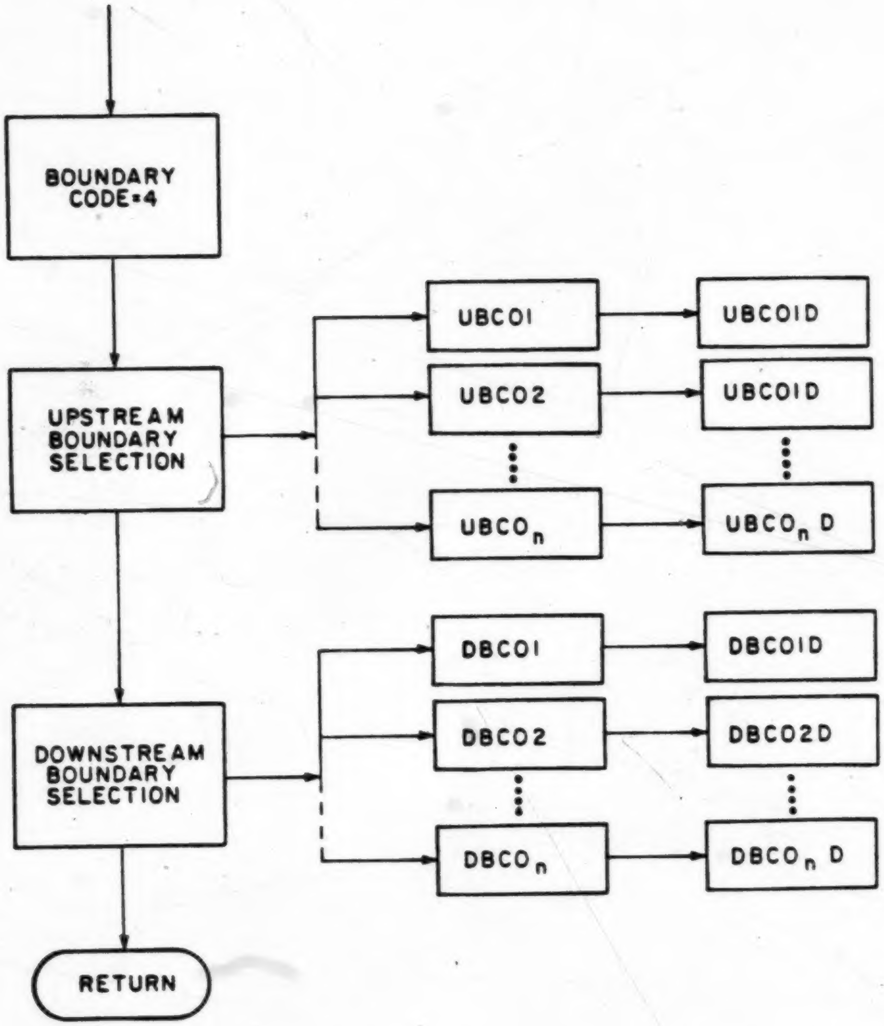

BOUNDARY CODE=4
UPSTREAM BOUNDARY SELECTION
UBCO1
UBCO2
UBCO$_n$
UBCO1D
UBCO1D
UBCO$_n$ D
DOWNSTREAM BOUNDARY SELECTION
DBCO1
DBCO2
DBCO$_n$
DBCO1D
DBCO2D
DBCO$_n$ D
RETURN

Figure 8.5 Detail flowchart of boundary condition preparation subroutine, BWPREP.

discharge into the next reach, the total discharge, and depth of water.

4. Calls subroutine DBC which selects the control subroutine which describes the downstream boundary of reach, DBC01, DBC02, etc., based on the second parameter of matrix DBCPR.
5. If the boundary code equals four the control subroutine will call a subroutine, DBC01D, DBC02D, etc., which will then perform the necessary downstream boundary parameter update for the same reach for which the backwater calculations have been performed. The parameters updated may include canal off-take adjustments so as to maintain previously specified deliveries.

8.3.2.2.7 Unsteady Flow Coefficient Calculations

The calculation of the initial coefficients which are specific to the particular upstream and downstream boundary conditions and used in the unsteady flow routing subroutine, HYDRT, are calculated using the subroutine COCALC. A flowchart of COCALC is shown in Figure 8.6. COCALC performs as follows:

1. Sets boundary code equal to three.
2. Calls subroutine UBC which selects the control subroutine which describes the upstream boundary UBC01, UBC02, etc., based on the second parameter of matrix UBCPR.
3. The control subroutine will call a subroutine, UBC01A, UBC02A, etc., if the boundary code equals three. This subroutine will initialize the coefficients corresponding to the upstream

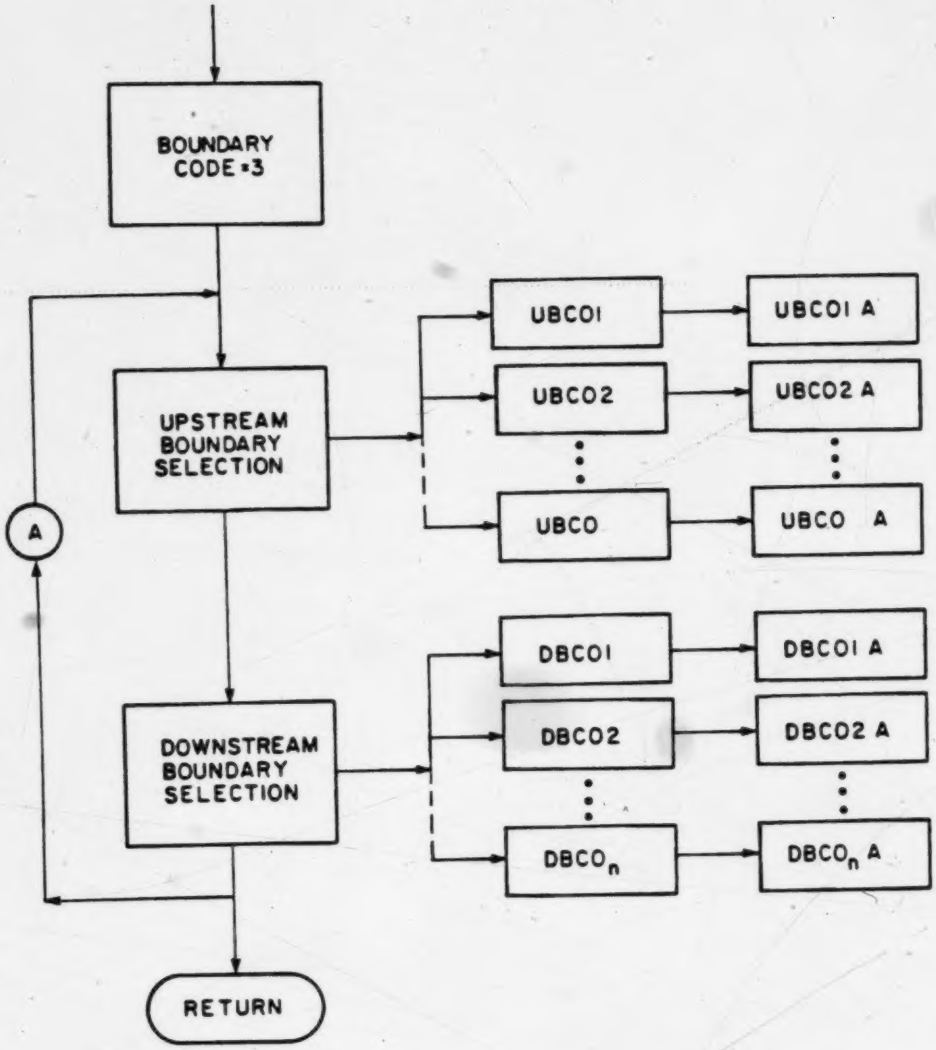

BOUNDARY CODE = 3
UPSTREAM BOUNDARY SELECTION
UBCO1
UBCO1 A
UBCO2
UBCO2 A
UBCO
UBCO A
DOWNSTREAM BOUNDARY SELECTION
DBCO1
DBCO1 A
DBCO2
DBCO2 A
DBCO$_n$
DBCO$_n$ A
A
RETURN

Figure 8.6 Detailed flowchart of unsteady flow coefficient calculation subroutine, COCALC.

boundaries of each reach and used by subroutine HYDRT to perform the unsteady flow routing procedures.

4. Calls subroutine DBC which selects the control subroutine which describes the downstream boundary, DBC01, DBC02, etc., based on the second parameter of matrix DBCPR.
5. The control subroutine will call a subroutine, DBC01A, DBC02A, etc., if the boundary code equals three. This subroutine will initialize the coefficients corresponding to the downstream boundaries of each reach and used by subroutine HYDRT to perform the unsteady flow routing procedures.
6. Steps 2 to 5 will be repeated for each reach.

8.3.2.2.8 Duration of Simulation Control Variable

The initialization of the duration of simulation control variable is performed. This variable keeps track of the actual duration the simulation has operated. It is later compared to the expected duration of study to determine if the operator wishes to continue or terminate the simulation.

8.3.2.2.9 Writing Reach Specific Statistics

The writing of the initial values of the reach specific variables is controlled by subroutine REPORT. A flowchart of REPORT is shown in Figure 8.7. REPORT performs as follows:

1. Report control index number nine is tested. If equal to one instantaneous and cumulative water balance data is written to a file named REPORT1. The following information is recorded:

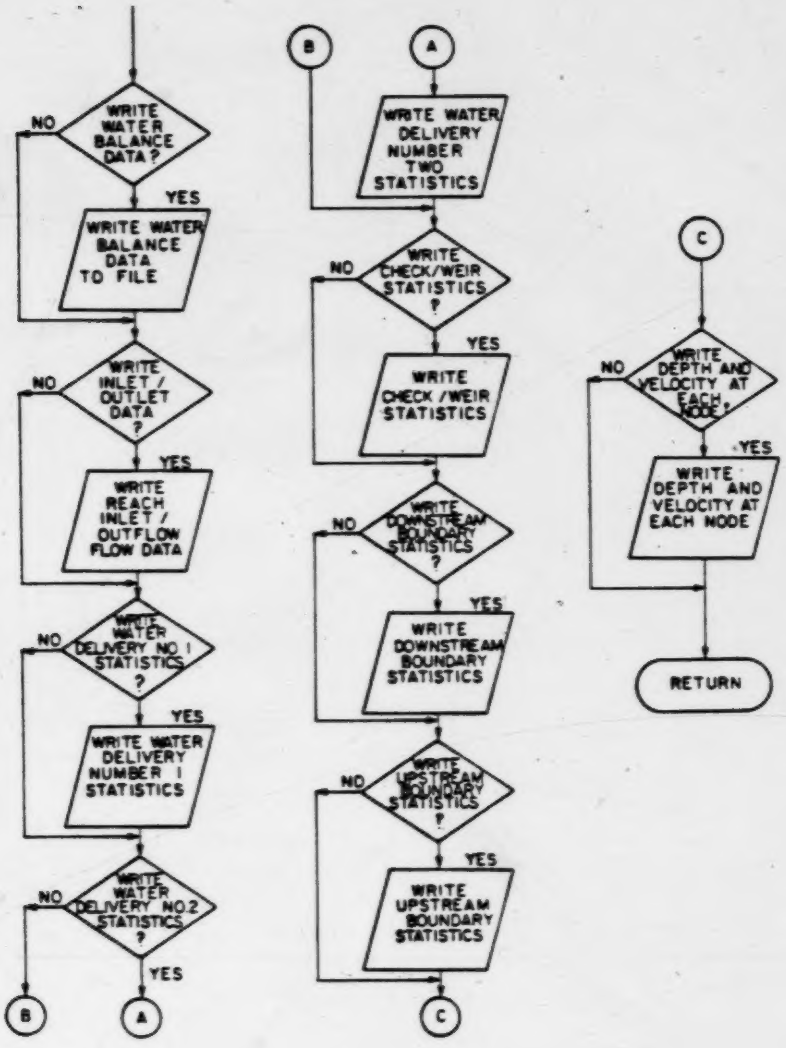

WRITE WATER BALANCE DATA ?
NO
YES
WRITE WATER BALANCE DATA TO FILE
WRITE INLET / OUTLET DATA ?
NO
YES
WRITE REACH INLET / OUTFLOW FLOW DATA
WRITE WATER DELIVERY NO 1 STATISTICS ?
NO
YES
WRITE WATER DELIVERY NUMBER 1 STATISTICS
WRITE WATER DELIVERY NO 2 STATISTICS ?
NO
YES
B
A
B
A
WRITE WATER DELIVERY NUMBER TWO STATISTICS
WRITE CHECK/WEIR STATISTICS ?
NO
YES
WRITE CHECK /WEIR STATISTICS
WRITE DOWNSTREAM BOUNDARY STATISTICS ?
NO
YES
WRITE DOWNSTREAM BOUNDARY STATISTICS
WRITE UPSTREAM BOUNDARY STATISTICS ?
NO
YES
WRITE UPSTREAM BOUNDARY STATISTICS
C
C
WRITE DEPTH AND VELOCITY AT EACH NODE ?
NO
YES
WRITE DEPTH AND VELOCITY AT EACH NODE
RETURN

Figure 8.7 Detailed flowchart of subroutine, REPORT.

- reach number,
- time,
- discharge at beginning of reach,
- discharge at end of reach,
- actual discharge to downstream reach,
- simulation discharge to downstream reach,
- rate of useful water delivery,
- storage in reach x 10^{-3},
- rate of seepage loss from entire reach,
- rate of evaporation loss from entire reach,
- rate of precipitation gain to entire reach,
- rate of other distributed lateral inflows or outflows to or from entire reach,
- minimum rate of flow to downstream reach,
- rate of seepage loss from reach per unit length of reach,
- rate of evaporation loss from reach per unit length of reach,
- rate of precipitation inflow to reach per unit length of reach,
- rate of other distributed inflow or outflow to or from the reach per unit length of reach,
- rate of useful flow to downstream reach,
- cumulative volume of inflow to reach,
- cumulative volume of flow at end of reach, (not to be confused with flow out of reach),

- cumulative volume of flow out of reach to downstream reach,
- cumulative volume of simulation flow to downstream reach,
- cumulative volume of useful water delivery,
- actual storage in reach x 10^{-3},
- cumulative volume of seepage losses from reach,
- cumulative volume of evaporation losses from reach,
- cumulative volume of precipitation gains to reach,
- cumulative volume of other distributed lateral inflow or outflow to or from reach,
- cumulative minimum volume of simulation outflow from reach to downstream reach, and,
- cumulative volume of useful flow to downstream reach.

2. Report control index number seven is tested. If equal to one, reach inlet and outlet flow conditions are written to a file named REPORT2. The information recorded includes:
    - reach number,
    - time,
    - depth at beginning of reach,
    - discharge at beginning of reach,
    - depth at end of reach, and
    - discharge at end of reach.

3. Report control index number four is tested. If equal to one, the water delivery number one statistics are written to a file named REPORT3. The information recorded includes:
    - reach number,
    - time,
    - water requirements,
    - actual water delivery,
    - upper tolerance limit to error in water delivery,
    - lower tolerance limit to error in water delivery,
    - useful water delivery,
    - maximum turnout opening,
    - actual turnout opening, and
    - depth of water at turnout.
4. Report control index number five is tested. If equal to one, the water delivery number two statistics are written to a file named REPORT4. The information recorded is the same as is recorded by REPORT3 on water delivery number one.
5. Report control index number three is tested. If equal to one, the check or weir statistics are written to a file named REPORT5. The following information is recorded:
    - reach number,
    - time,
    - current height of check or weir,
    - actual discharge to downstream reach,
    - observed discharge to downstream reach,
    - minimum discharge to downstream reach, and

- simulation discharge to downstream reach.

6. Report control index number eleven is tested. If equal to one, the entire matrix describing the downstream boundary for that reach is written to a file named DBCPAR. The information recorded is specific to the type of downstream boundary being considered. The individual downstream boundaries considered by the ICSS model are discussed in a later section.
7. Report control index number twelve is tested. If equal to one, the entire matrix describing the upstream boundary for that reach is written to a file named UBCPAR. The information recorded is specific to the type of upstream boundary being considered. The individual upstream boundaries considered by the ICSS model are described in a later section.
8. Report control index number six is tested. If equal to one, the depth and velocity at each node in the reach is recorded in a file named REPORT6.

8.3.2.3 Operations

The operations of the operable boundaries of each reach are controlled by a subroutine named OPERAT. A flowchart of OPERAT is shown in Figure 8.8. The subroutine performs as follows:

1. Sets boundary code equal to one.
2. Displays message on terminal indicating that operations are permitted and requests that the operator either input the number of the reach which contains the structure he wishes to operate or indicate that he wishes to terminate operations.

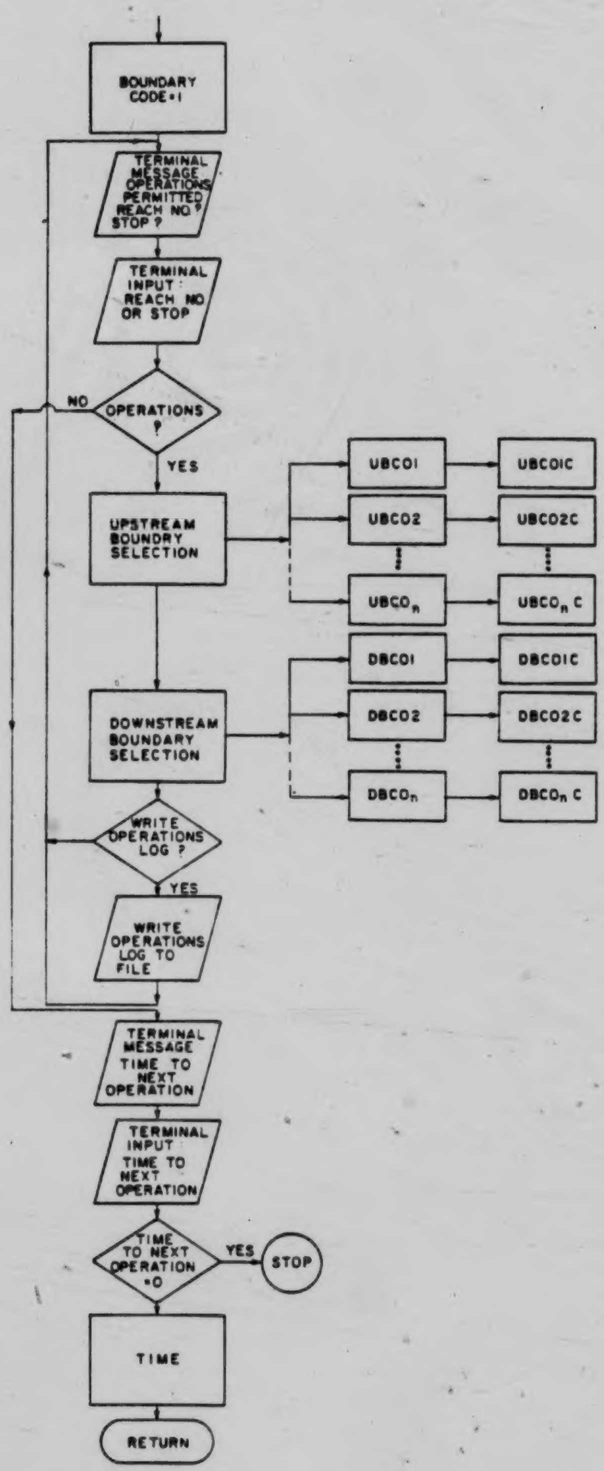

BOUNDARY CODE=1
TERMINAL MESSAGE OPERATIONS PERMITTED REACH NO? STOP?
TERMINAL INPUT: REACH NO OR STOP
NO
OPERATIONS ?
YES
UPSTREAM BOUNDRY SELECTION
UBCO1
UBCO1C
UBCO2
UBCO2C
UBCOn
UBCOn C
DOWNSTREAM BOUNDARY SELECTION
DBCO1
DBCO1C
DBCO2
DBCO2C
DBCOn
DBCOn C
WRITE OPERATIONS LOG ?
YES
WRITE OPERATIONS LOG TO FILE
TERMINAL MESSAGE TIME TO NEXT OPERATION
TERMINAL INPUT TIME TO NEXT OPERATION
TIME TO NEXT OPERATION =0
YES
STOP
TIME
RETURN

Figure 8.8 Detailed flowchart of operations subroutine, OPERAT.

3. The operator either inputs the number of the reach he wishes to operate or he inputs a number code which indicates his desire to terminate operations.
4. The operator's input is checked.
5. If he wishes to terminate the session he is asked when he wishes to perform operations again.
6. The operator will input the time to next operation.
7. The operator's input will be checked. If equal to 0.0, the simulation terminates. If greater than zero, simulation continues and subroutine TIME is called which indicates on the terminal the time the operations were performed.
8. If the operator had wished to continue operations, he would have input the number of the reach containing the boundaries he wished to operate or simply examine.
9. Subroutine UBC is called. UBC selects the control subroutine which describes the upstream boundary for reach, UBC01, UBC02, etc., based on the second parameter of matrix UBCPR.
10. The control subroutine will call subroutine, UBC01C, UBC02C, etc., on the basis that the boundary code equals one. These subroutines will then present necessary information specific to the boundary to the operator on the terminal which will allow him to operate the boundary if it is operable. When the operator is finished reviewing or adjusting the boundary he will indicate so and control will return to subroutine OPERAT.

11. Subroutine DBC is called. DBC selects the control subroutine which describes the downstream boundary for reach, DBC01, DBC02, etc., based on the second parameter of DBCPR.
12. The control subroutine will call subroutine DBC01C, DBC02C, etc., on the basis that the boundary code equals one. These subroutines will then present information to the operator, similar to 10., on the terminal and allow him to operate the boundary if it is operable.

    When the operator is finished reviewing or adjusting the boundary, he will indicate so and control will return to OPERAT.
13. The fact that the operator has reviewed and possibly operated the boundaries of the reach may be written to file depending on how report control index number eight is initialized.
14. The operations continue until the operator indicates he wishes to stop by entering the appropriate code in Step 3.

8.3.2.4 Hydrological Calculations

The hydrological calculations segment consists entirely of the subroutine HYDROL, which is thoroughly described in Section 8.3.2.2.5.

8.3.2.5 Unsteady Flow Calculations

8.3.2.5.1 Function

The unsteady flow calculation segment performs the following functions:

1. Calculates the depth and velocity using subroutine HYDRT at each node of a single reach as the reach flow conditions and the upstream and downstream boundary conditions vary with time.
2. Writes to file the current values of operator selected reach specific variables using subroutine REPORT discussed in Section 8.3.2.2.9.
3. Steps 1 and 2 are repeated for each reach of the canal starting with the most upstream reach.

Figure 8.9(a) illustrates how these functions are related.

8.3.2.5.2 Hydraulic Routing - Unsteady Flow Conditions

The hydraulic routing under unsteady flow conditions is performed by subroutine HYDRT. A flowchart of HYDRT is shown in Figure 8.9(b). The specific equations being solved are

$$\frac{A}{B}\frac{\partial V}{\partial x} + V\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} - \frac{(p-i)}{B} = 0 \qquad (8.4)$$

and

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} + g(S_f - S_o) + \frac{V(p-i)}{A} = 0 \qquad (8.5)$$

where B = the width of channel at the water surface;

t = time; and,

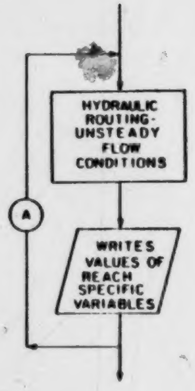

HYDRAULIC ROUTING-UNSTEADY FLOW CONDITIONS
WRITES VALUES OF REACH SPECIFIC VARIABLES
A

Figure 8.9(a) Flowchart of unsteady flow calculation segment.

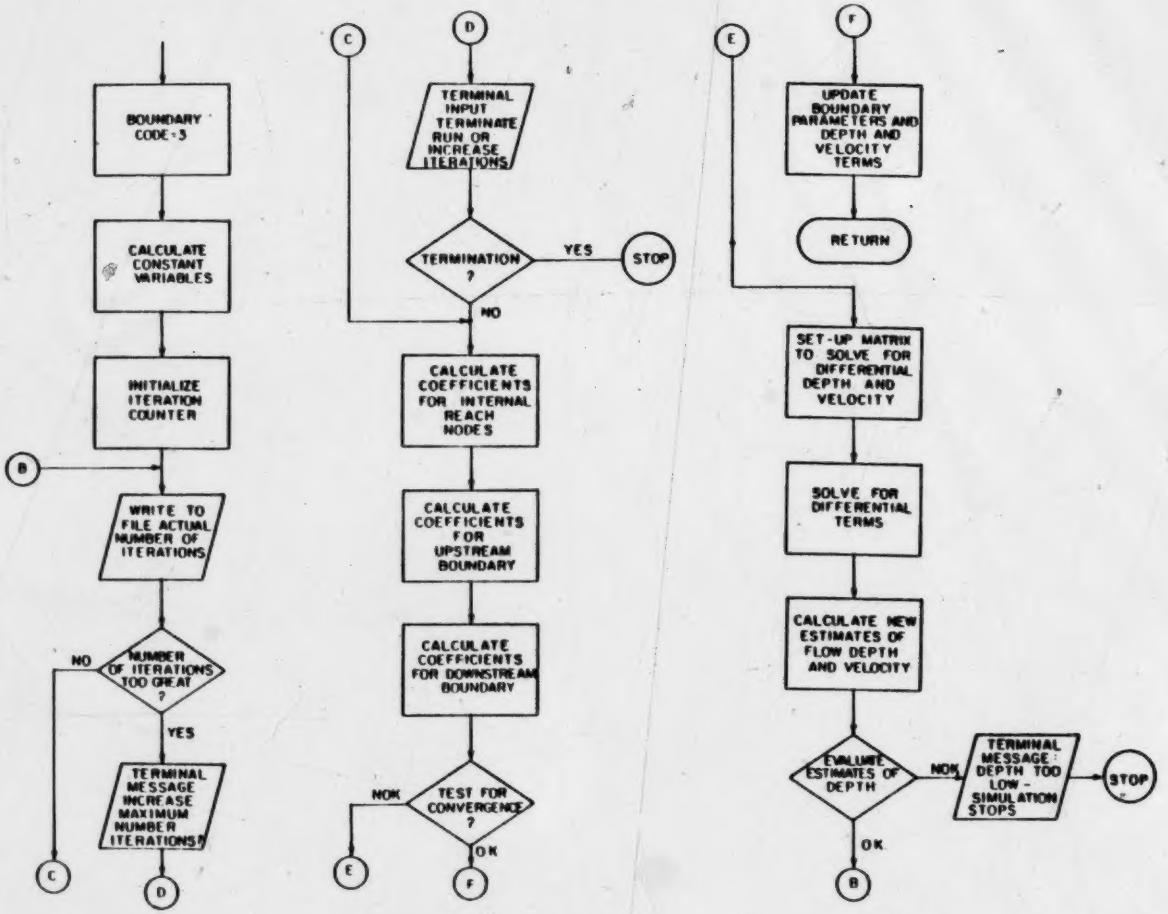

BOUNDARY CODE-3
CALCULATE CONSTANT VARIABLES
INITIALIZE ITERATION COUNTER
B
WRITE TO FILE ACTUAL NUMBER OF ITERATIONS
NUMBER OF ITERATIONS TOO GREAT ?
NO
YES
TERMINAL MESSAGE INCREASE MAXIMUM NUMBER ITERATIONS?
C
D
C
D
TERMINAL INPUT TERMINATE RUN OR INCREASE ITERATIONS
TERMINATION ?
YES
STOP
NO
CALCULATE COEFFICIENTS FOR INTERNAL REACH NODES
CALCULATE COEFFICIENTS FOR UPSTREAM BOUNDARY
CALCULATE COEFFICIENTS FOR DOWNSTREAM BOUNDARY
TEST FOR CONVERGENCE ?
NOK
OK
E
F
E
F
UPDATE BOUNDARY PARAMETERS AND DEPTH AND VELOCITY TERMS
RETURN
SET-UP MATRIX TO SOLVE FOR DIFFERENTIAL DEPTH AND VELOCITY
SOLVE FOR DIFFERENTIAL TERMS
CALCULATE NEW ESTIMATES OF FLOW DEPTH AND VELOCITY
EVALUATE ESTIMATES OF DEPTH
NOK
TERMINAL MESSAGE: DEPTH TOO LOW - SIMULATION STOPS
STOP
OK
B

Figure 8.9(b) Detailed flowchart of subroutine, HYDRT.

the other variables are defined as in Equations (8.1), (8.2) and (8.3). Equations (8.4) and (8.5) are solved using an implicit finite difference solution described in detail in Appendix A.

HYDRT performs as follows:

1. The boundary code is set equal to three.
2. All variables, which remain constant during the iteration process used to calculate the depth and velocity at each node of the reach one hydraulic time interval later, are calculated.
3. The variable used to count the number of iterations required to calculate the depth and velocity at each node of the reach one hydraulic time interval later is initialized.
4. The actual number of iterations is written to a file.
5. The actual number of iterations is compared to the maximum number specified within the subroutine. If the number of iterations is too great, a message is displayed on the terminal informing the model operator that a possible instability may exist.
6. The operator may wish to increase the maximum number of iterations in order to continue the simulation or he may wish to terminate it.
7. If the actual number of iterations is small or if the model operator elects to continue the simulation, the unsteady flow coefficients for each of the reach internal nodes are calculated.

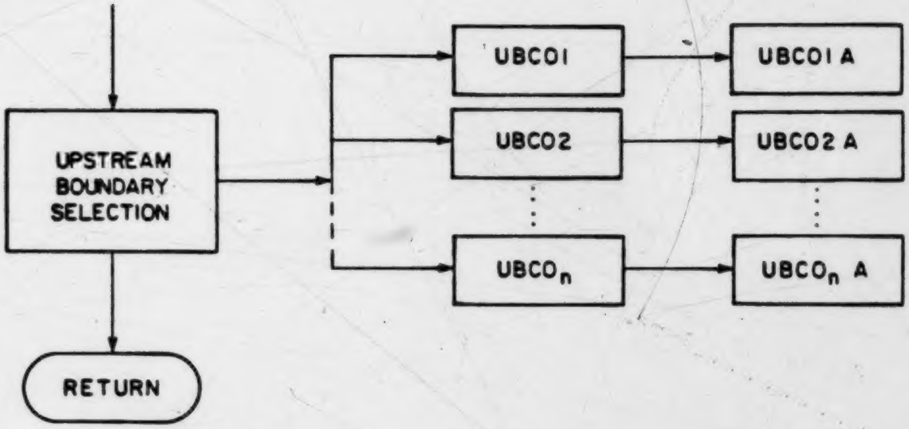

UPSTREAM BOUNDARY SELECTION
UBCO1
UBCO2
UBCO$_n$
UBCO1 A
UBCO2 A
UBCO$_n$ A
RETURN

Figure 8.10(a) Detailed flowchart of unsteady flow routing coefficient calculation routine, upstream boundaries.

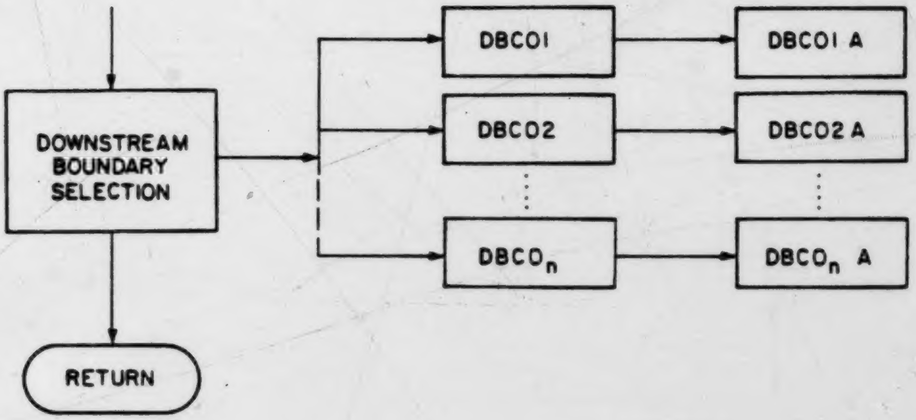

DOWNSTREAM BOUNDARY SELECTION
DBCO1
DBCO2
DBCO$_n$
DBCO1 A
DBCO2 A
DBCO$_n$ A
RETURN

Figure 8.10(b) Detailed flowchart of unsteady flow routing coefficient calculation routine, downstream boundaries.

8. The unsteady flow coefficients for the upstream and downstream boundaries are calculated by calling subroutines UBC and DBC, which select the appropriate subroutines describing the boundaries. These processes are illustrated in Figures 8.10(a) and (b). The selected subroutines will in turn call other subroutines, based on the boundary code being equal to one, which perform the actual calculations.
9. Unsteady flow coefficient number five, which would equal zero if perfect convergence is achieved, is compared against an acceptable limit previously specified by the model operator as a simulation control variable.
10. If the convergence is not acceptable, a matrix is formed of the unsteady flow coefficients which enables the solution of the required differential changes in velocity and depth estimates. (The initial estimate of velocity and depth are the values calculated for the previous hydraulic time interval.)
11. The matrix of unsteady flow coefficients is solved using subroutine GAUSEL.
12. New estimates of depth and velocity are generated.
13. The depth estimates are tested to ensure that they are not too small or less than zero. If they are, a terminal message informs the model operator of the situation and the simulation stops. If the depth estimates are acceptable calculations are continued (at location B) until the test for convergence is acceptable.

14. The estimates of depth and velocity are used to update the upstream and downstream boundary parameters and reinitialize the depth and velocity values. These operations are performed by subroutine BCPPEP.

8.3.2.5.3 Subroutine BCPREP

A detailed flow chart of subroutine BCPREP is shown in Figure 8.11. As indicated earlier, the subroutine updates the upstream and downstream boundary parameters and reinitializes the depth and velocity values associated with previously specified reach. BCPREP performs as follows:

1. Sets boundary code equal to two.
2. Calls subroutine UBC which selects the control subroutine which describes the upstream boundary, UBC01, UBC02, etc., based on the second parameter of matrix UBCPR.
3. The control subroutine will call a subroutine, UBC01B, UBC02B, etc., on the basis that the boundary code equals two. This subroutine will update the matrix containing the parameters describing the particular upstream boundary of the reach under consideration.
4. Calls subroutine DBC which selects the control subroutine which describes the downstream boundary, DBC01, DBC02, etc., based on the second parameter of DBCPR.
5. The control subroutine will call a subroutine DBC01B, DBC02B, etc,. on the basis that the boundary code equals two. This subroutine will update the matrix containing the parameters describing the particular downstream boundary of the reach

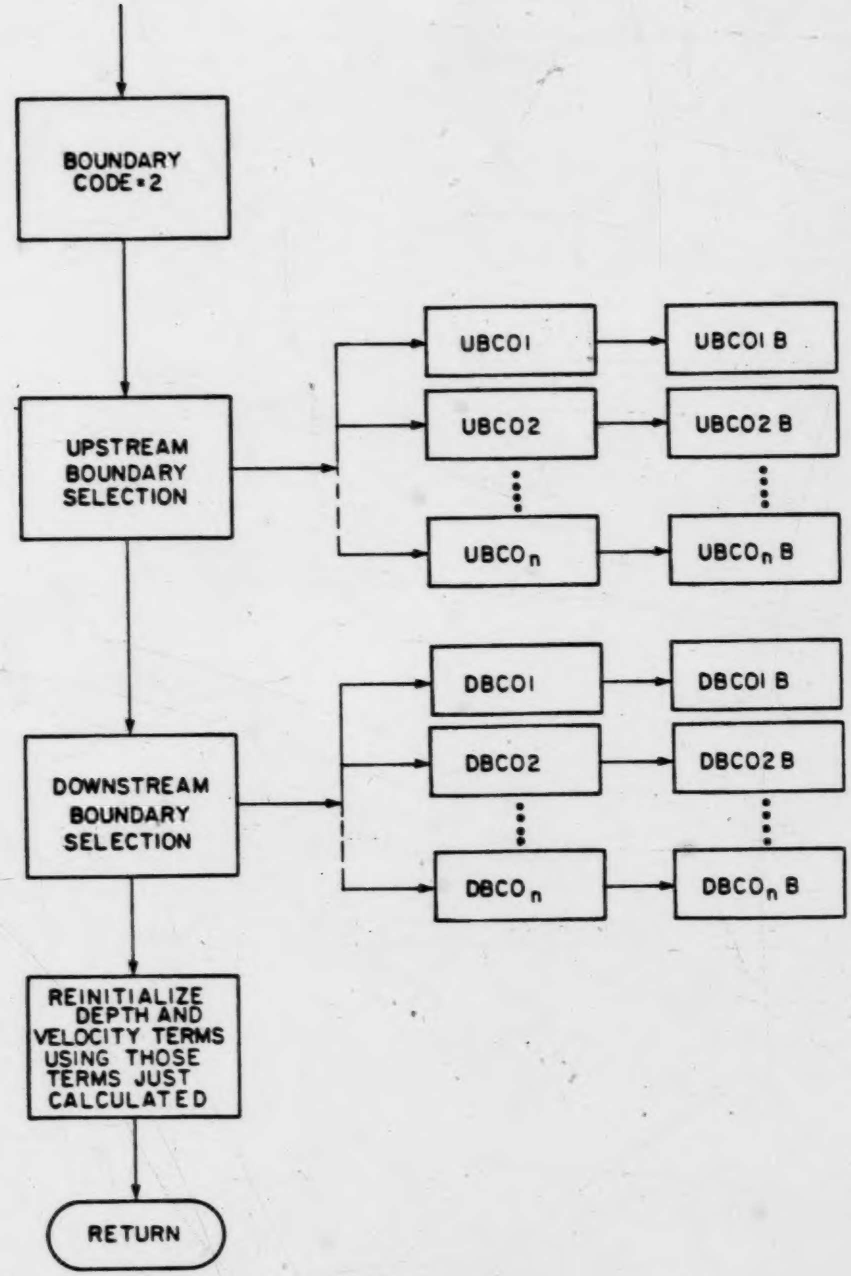

BOUNDARY CODE = 2
UPSTREAM BOUNDARY SELECTION
UBCO1
UBCO2
UBCOn
UBCO1 B
UBCO2 B
UBCOn B
DOWNSTREAM BOUNDARY SELECTION
DBCO1
DBCO2
DBCOn
DBCO1 B
DBCO2 B
DBCOn B
REINITIALIZE DEPTH AND VELOCITY TERMS USING THOSE TERMS JUST CALCULATED
RETURN

Figure 8.11 Detailed flowchart of subroutine, BCPREP.

under consideration.

8.3.3 Simulation of the Operational and Hydraulic Characteristics of Hydraulic Control Structures

The ICSS model provides the model operator considerable flexibility in the inclusion, selection, and application of algorithms which describe the operational and hydraulic characteristics of individual or groups of hydraulic control structures that define the beginning and termination of each reach. However, the following two restrictions apply:

1. All hydraulic control structures considered, individually or in groups, must exhibit either upstream or positive hydraulic control.
2. Only fixed or manually operated hydraulic structures may be considered.

The characteristics of individual or groups of hydraulic control structures, which define the beginning and end of each reach, are considered in the corresponding UBCO_n and DBCO_n series of subroutines, their associated subroutines, UBCO_n A, B, C, and D and DBCO_n A, B, C, and D, and the matrices, UBCPR and DBCPR. The second parameter of the matrices UBCPR and DBCPR allow the subroutines UBC and DBC to select the appropriate control subroutine, UBCO_n and DBCO_n. The current value of the boundary code will determine which of the A, B, C, or D subroutines will be called.

Subroutines UBC and DBC can presently accommodate a maximum of four types of upstream and downstream boundaries. Detailed flowcharts of these subroutines are shown in Figures 8.12(a) and (b) where it will be noted that the UBC and DBC subroutines can be easily expanded to consider as many types of boundaries as required. The present version of the ICSS model includes two types each of upstream and downstream boundary which are considered in greater detail in Chapter 9.

The flow chart of typical subroutine $UBCO_n$ is shown in Figure 8.13. It is noted that the function of $UBCO_n$ is to call the appropriate A, B, C, or D subroutines which perform the following four functions:

1. Subroutine $UBCO_nA$ calculates the unsteady flow coefficients.
2. Subroutine $UBCO_nB$ updates the UBCPR parameters four, five and ten corresponding to total flow rate at reach inlet, depth at reach inlet and rate of inflow from upstream reach.
3. Subroutine $UBCO_nC$ facilitates the interactive operation of the boundary.
4. Subroutine $UBCO_nD$ initializes UBCPR parameters four, five, and ten and may initialize DBCPR parameters four, five, and ten for the reach immediately upstream. (Note that DBCPR parameters are defined similar to the UBCPR parameters.)

The flowchart and functions for a typical subroutine $DBCO_n$ are similar to $UBCO_n$ as are subroutines $DBCO_nA$, $DBCO_nB$, and $DBCO_nC$ similar to $UBCO_nA$, $UBCO_nB$, and $UBCO_nC$ respectively. Subroutine $DBCO_nD$, however, may initialize DBCPR parameters four, five, eighteen, and twenty plus a variety of other parameters to ensure that the flow depth and flow rate characteristics at the end of the reach correspond to the adjustable physical characteristics of the hydraulic control structures being simulated.

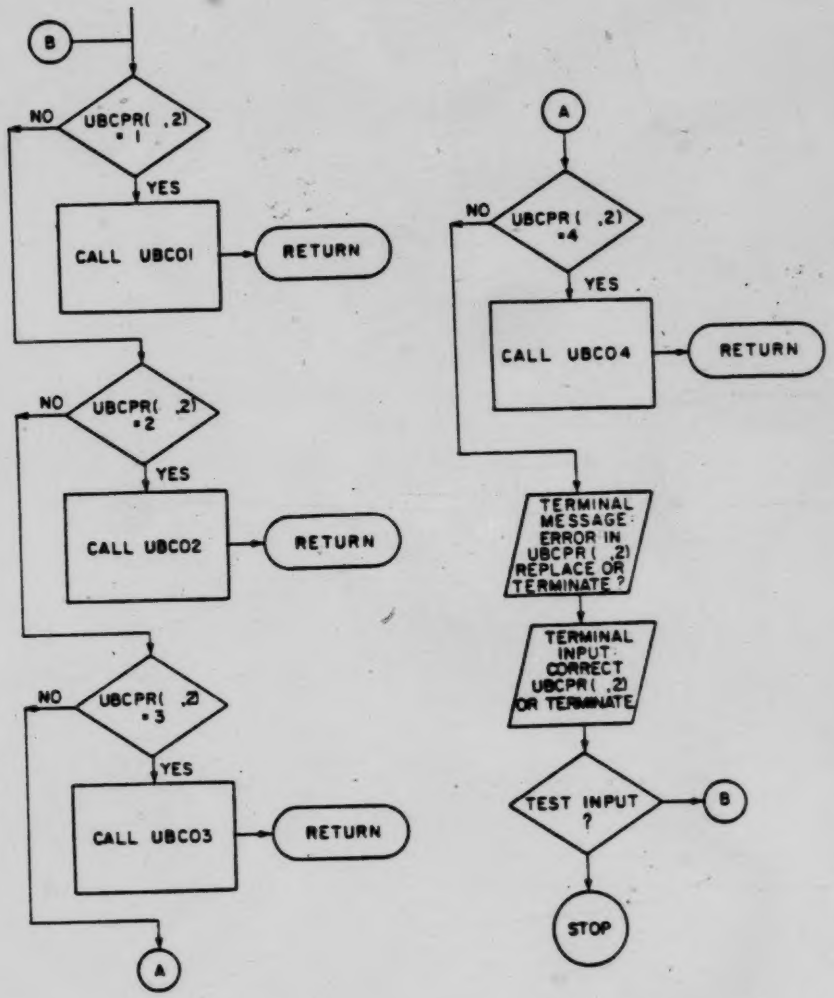

B
UBCPR(,2) =1
NO
YES
CALL UBCO1
RETURN
UBCPR(,2) =2
NO
YES
CALL UBCO2
RETURN
UBCPR(,2) =3
NO
YES
CALL UBCO3
RETURN
A
A
UBCPR(,2) =4
NO
YES
CALL UBCO4
RETURN
TERMINAL MESSAGE: ERROR IN UBCPR(,2) REPLACE OR TERMINATE ?
TERMINAL INPUT: CORRECT UBCPR(,2) OR TERMINATE
TEST INPUT ?
B
STOP

Figure 8.12(a) Detailed flowchart of subroutine, UBC.

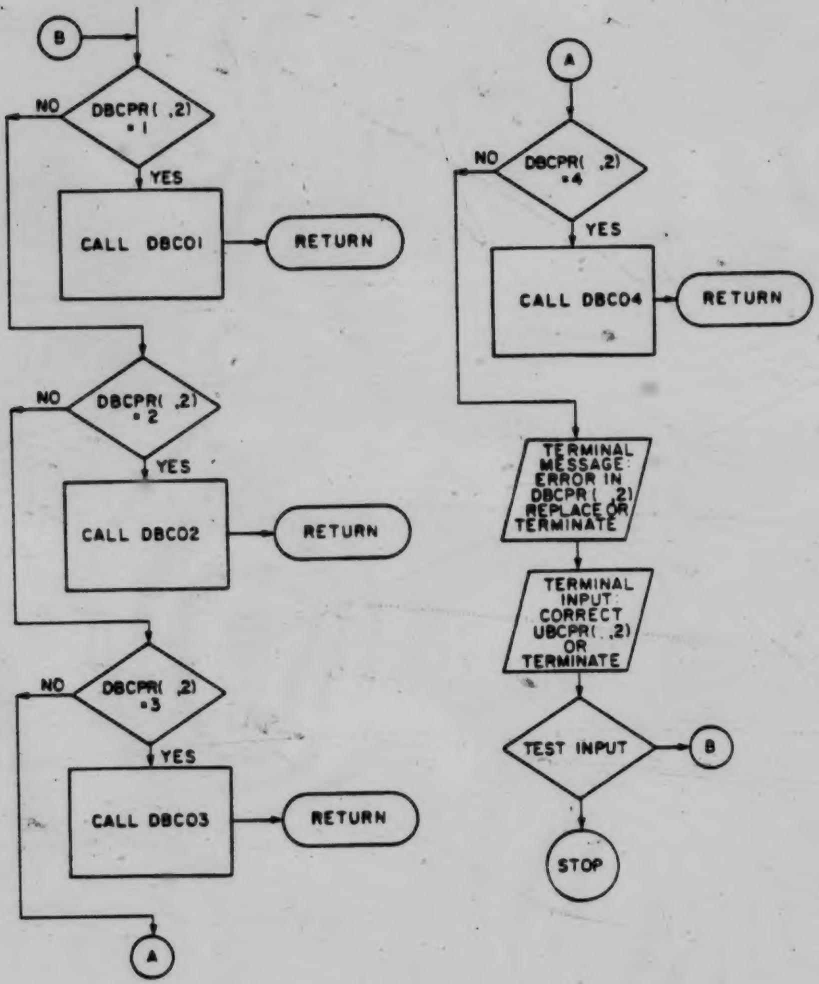

B
DBCPR(,2) =1
NO
YES
CALL DBC01
RETURN
DBCPR(,2) =2
NO
YES
CALL DBC02
RETURN
DBCPR(,2) =3
NO
YES
CALL DBC03
RETURN
A
A
DBCPR(,2) =4
NO
YES
CALL DBC04
RETURN
TERMINAL MESSAGE: ERROR IN DBCPR(,2) REPLACE OR TERMINATE
TERMINAL INPUT: CORRECT UBCPR(,2) OR TERMINATE
TEST INPUT
B
STOP

Figure 8.12(b) Detailed flowchart of subroutine, DBC.

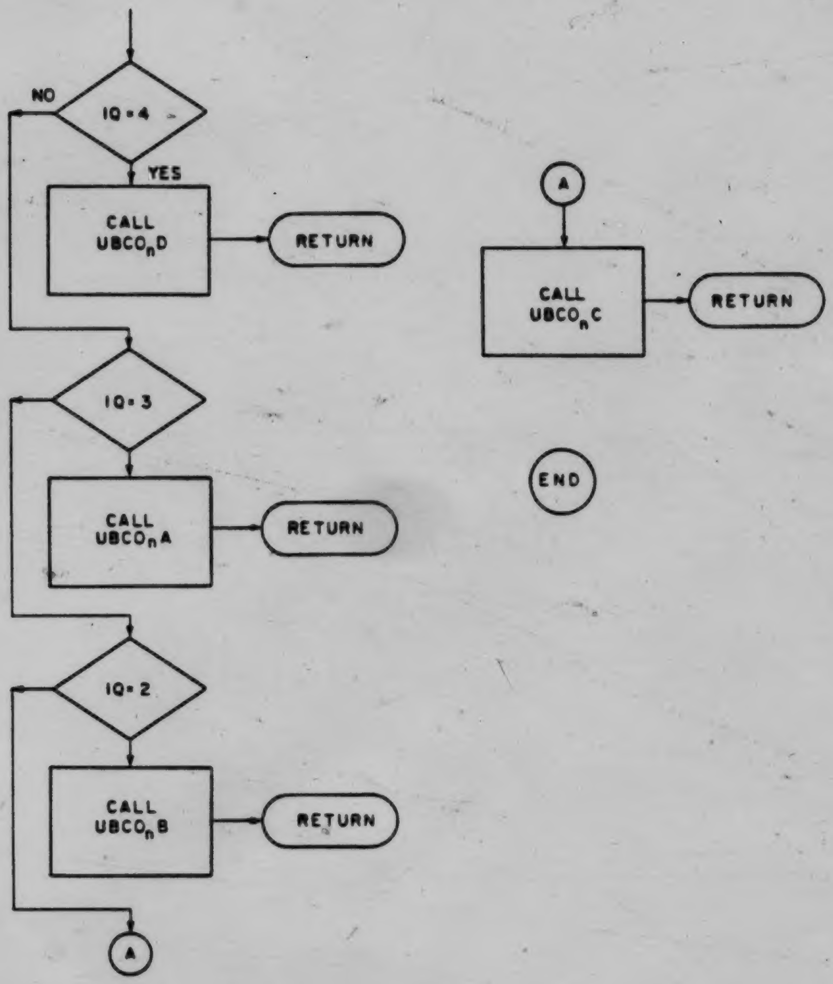

NO
IQ=4
YES
CALL UBCO$_n$D
RETURN
IQ=3
CALL UBCO$_n$A
RETURN
IQ=2
CALL UBCO$_n$B
RETURN
A
A
CALL UBCO$_n$C
RETURN
END

Figure 8.13 Detailed flowchart of typical subroutine, UBCO$_n$.

The matrices UBCPR and DBCPR provide the model operator with the opportunity to dimension or otherwise characterize the individual types of upstream and downstream boundaries. For example, a particular type of downstream boundary, DBC02, may be selected by the canal operator to terminate each of the reaches of a canal he is simulating. The physical dimensions or flow coefficients may vary with each reach but the same collection of subroutines, DBC02, DBC02A, DBC02B, DBC02C, and DGC02D will be used to simulate the similar groups of hydraulic structures.

8.4 Summary of ICSS Model's Present Capabilities

In its present form the ICSS Model can simulate the flow of water through prismatic channels of variable cross-sectional shape and slope and hydraulic roughness. Distributed water losses or gains such as precipitation, seepage, evaporation and surface drainage may be specified and held constant during individual simulations or algorithms may be included which allow these losses or gains to vary. In this version of the Model seepage losses may be held constant or allowed to vary according to the Moritz equation. Similarly, hydraulic roughness may be specified and held constant or algorithms may be included which allow it to vary.

Though any number of individual channel reaches may be simultaneously considered, provision is made to consider a maximum of ten. This limit may be altered through the appropriate dimension statements.

Each reach must be bounded by a variety of structure groups which include any number or variation of the following components:

- Types 1-8 and 17-24 diversion structures
- Types 1-4, 9-16 and 21-24 depth-control structures
- canal-associated specialty structures exhibiting positive or upstream hydraulic control

In the present version of the Model, options available include one variation of canal inlet structure and two variations of canal outlet structure groups. From the operational standpoint the inlet structure may be considered either a Type 1 diversion structure or a Type 5 diversion structure operating with constant upstream depth. The first variation of canal outlet structure included is a vertical weir-like drop-structure. The second variation of canal outlet structure is a Type 1 weir-like depth-control structure combined with a drop-structure and a Type 1 orifice-like diversion structure.

Because the Model is executed interactively virtually any form of farm water demand may presently be considered.

The interactive feature also facilitates both upstream and downstream forms of manual operation. Upstream and downstream automatically operated systems may also be simulated within the existing model structure. This would require the development of algorithms describing the operation of those structure groups the investigator wishes to consider.

7

8.5 Discussion

The structure and version of the ICSS Model presented provides a means with which to evaluate the performance of a variety of irrigation conveyance systems, which may be operated using a variety of methods, to satisfy several types of water demand. The ICSS Model is not "complete" nor will it likely ever be complete. Rather, the model contains the basic structure which will enable its continuous development to consider the tremendous variety of irrigation conveyance systems presently in existence or yet to be designed. (For example, the techniques that enable consideration of hydraulic control structures which exhibit downstream and upstream/downstream hydraulic control and automatic and fixed-automatic structures are known and will be incorporated into a future version of the model.)

The model has not been calibrated using field data. It is difficult to imagine how it could be generally calibrated except through the piece-wise examination of the individual techniques the model uses in its simulations. Certainly the model could be calibrated if actual systems were being simulated for which adequate field data were available. However, this calibration would be valid only for the specific system studied. Each system being simulated would require the recalibration of the model to ensure the physical reality was being reproduced. This process would only be possible if the system was already in existence. It is apparent that it is an excessive restriction on the model's application to require that it be calibrated to each system being considered. The philosophy inherent to a restriction of this nature is opposite to accepted engineering design approaches. However, it is recognized that confidence in the model's application will only be gained through successful calibrations of the type described or the development of successful conveyance system designs and operating procedures.

Though considerable attention was paid to the incorporation of high quality (accurate) simulation procedures, other quality and proven procedures and techniques are likely available now or will be available in the future. It was for this reason that the model was structured in modular form. For example, if an investigator wishes to use another technique to simulate seepage losses, it may be incorporated and used without adversely affecting the remainder of the model's structure.

The interactive execution feature of the model significantly simplifies the simulation of the almost innumerable forms of irrigation conveyance system operations which one may wish to examine. The alternative to interactive execution is the development of individual subroutines which would simulate each form of manual operation of interest and executing the program in batch mode. This exercise has been examined thoroughly and in the opinion of the writer is not justified considering the tremendous amount of additional effort required and the increasing availability of inexpensive and relatively powerful computer systems.

Finally, the model may be expanded to consider as many individual reaches, parameters, indices, etc., as the investigator wishes by simply increasing the appropriate dimension statements. The current dimensioning reflects the limit of current analytical requirements and budget. Unnecessarily large matrices will needlessly increase computer requirements and costs.

CHAPTER 9

DEMONSTRATION OF THE IRRIGATION CONVEYANCE SYSTEM SIMULATION MODEL

9.1 Introduction

The purpose of this demonstration is to illustrate how the ICSS Model can be used to simulate the hydraulics and operation of specific irrigation conveyance systems and to demonstrate how the simulation results may be used to evaluate potential system performance.

The specific objectives of this demonstration are:

1. to illustrate how the ICSS Model is applied;
2. to demonstrate the methods for determining the analytical integrity of the model; and
3. to demonstrate the utility of the ICSS Model for purposes of evaluating irrigation conveyance system efficiency.

9.2 Description of Study Canal

The conveyance system selected for study and simulation is a distributary canal typical of those found in Southern Alberta, Canada. A plan view of the study canal is shown in Figure 9.1. The canal is of the earth canal type and is therefore subject to seepage losses and weed infestations. It has three reaches. Water flow into the canal is controlled by either a Type 1 diversion structure or by a Type 5 diversion structure with fixed upstream flow conditions, the effect on canal operation is similar. (See Section 4.2.3.1.) The water depth at the downstream end of each reach is controlled by a depth-control

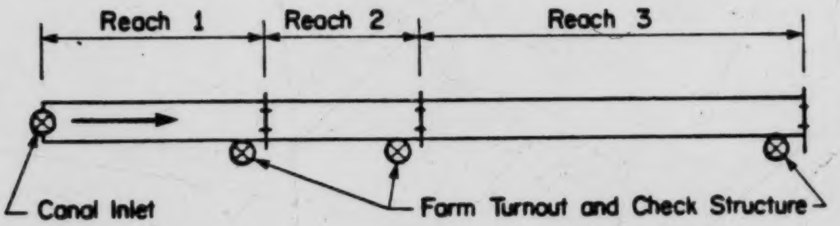

Reach 1
Reach 2
Reach 3
Canal Inlet
Farm Turnout and Check Structure

Figure 9.1 Plan view of study canal.

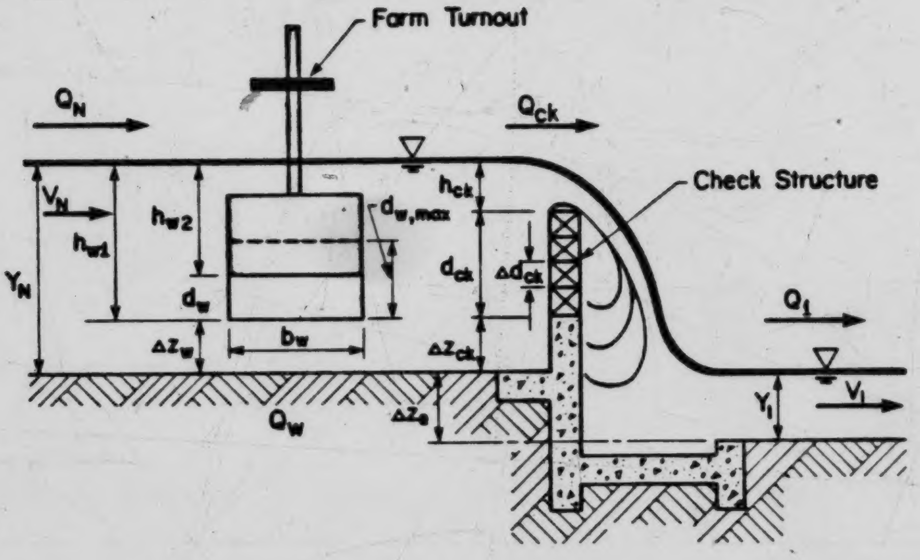

Farm Turnout
Q_N
V_N
Y_N
h_w1
h_w2
d_w
Δz_w
b_w
d_w,max
Q_ck
h_ck
d_ck
Δd_ck
Δz_ck
Check Structure
Q_1
Y_1
V_1
Q_w
Δz_e

Figure 9.2 Detail of farm turnout and check-structure.

("check") structure. (See Section 4.4.3.1.) Water is delivered to the farmers through farm turnouts which are Type 5 diversion structures. The farm water demand being satisfied may be described as being collaboratively controlled unlimited frequency, collaboratively controlled limited rate, and collaboratively controlled unlimited duration. This corresponds to the Repogle and Merriam Type 5, restricted arranged schedule. The canal is operated using manual upstream control.

Specific details of the farm turnout and check structure are shown in Figure 9.2. Water depth upstream of the depth-control structure is controlled by inserting or removing stop-logs of width, Δd_{ck}. The minimum height of the check structure is Δz_{ck}. Flow past the check structure, Q_{ck}, is assumed to be described by the weir equation with the velocity head assumed negligible so that Q_{ck} depends only on h_{ck} and the width of the check, b_{ck}. Note that there is a change in canal invert elevation, Δz_e, across the check-structure. The check has a maximum height of $h_{ck,max}$. Flow through the farm turnout, Q_w, is controlled by opening or closing the orifice opening, d_w. The maximum orifice opening is $d_{w,max}$ and the orifice width is b_w. The orifice opening is Δz_w above the canal invert. Flow through the orifice is assumed to be governed by the equation for an orifice with a free outlet if the water depth, y_N, is greater than the sum of Δz_w and d_w. Otherwise, flow through the orifice is assumed to be governed by the weir formula.

The flow rate immediately upstream of the farm turnout is equal to Q_N and the average velocity and depth are V_N and y_N respectively. The flow rate immediately below the check structure, Q_1, is equal to the flow rate past the check-structure, Q_{ck}. The water velocity and depth downstream of the check structure are V_1 and y_1 respectively.

The inlet structure to the canal was identified as a Type 1 boundary condition (as far as the ICSS Model was concerned). Since this structure could exist at the upstream end of canal reach number one, only the subroutine describing the upstream boundary condition needed to be written (see Section 8.3.3).

The farm turnout and check structure were combined to form a single hydraulic structure group identified as a Type 2 downstream boundary condition depending on the perspective of the reach in question. For example, reach number one in Figure 9.1 would have a Type 1 upstream boundary condition and a Type 2 downstream boundary condition. Reaches number two and three would have Type 2 upstream boundary condition and Type 2 downstream boundary condition.

The canal is earth lined and is assumed subject to seepage losses as predicted by the Moritz formula discussed in Appendix E. The canal is trapezoidal in shape and has a mild slope. The seepage rate is predicted using the average cross-sectional area of the canal reach and is assumed to be constant along the entire reach. The errors introduced by this approximation are assumed to be insignificant compared to the errors inherent in the use of the formula itself.

9.3 Evaluation of the Analytical Integrity of the Model

The analytical integrity of the ICSS Model was evaluated in the following three ways in the application just described:

1. comparison of the longitudinal water depth profile as calculated using the steady and unsteady hydraulic routing techniques;
2. evaluation of the water balance error in a variety of flow experiments with and without seepage losses; and,
3. sensitivity analysis of parameters affecting the unsteady flow calculations.

The results of all three evaluations indicated that the analytical techniques used to estimate the hydraulic conditions in the canal were accurate.

9.3.1 Comparison of Longitudinal Water Depth Profiles as Calculated Using Steady and Unsteady Hydraulic Routing Procedures

The longitudinal water depth and velocity profiles as calculated by the steady and unsteady hydraulic routing techniques (see Appendices A and B) were compared in two ways. First, if the two routing techniques produced similar results, the unsteady hydraulic routing procedure should not attempt significant modification to the estimate of the flow profile as provided by the steady hydraulic routing procedure. This was found to be the case in all instances even after several hours of simulation with and without seepage losses. The unsteady routing procedure did make some modification to water depth and velocity but these were always found to be less than one millimeter in the depth and two millimeters per second in the velocity representing relative errors of less than one-half of one percent. Secondly, if allowed to achieve steady-state conditions, the unsteady flow routing procedure would always converge to the results predicted by the steady flow routing procedure. Since it is unreasonable to expect that two so different hydraulic routing procedures should both generate identical errors it can be assumed that they represent a mutual accuracy check which indicates that both techniques predict the steady-state longitudinal depth/velocity profile correctly.

9.3.2 Evaluation of Water Balance Error in the Hydraulic Routing Procedures

Examination of water balance data produced by the ICSS Model indicate that errors in the water balance are consistently less than one percent and often less than one-tenth of one percent. This result was anticipated considering the characteristics of the steady and unsteady hydraulic routing techniques incorporated in the model.

9.3.3 Sensitivity Analysis of Parameters Affecting Unsteady Flow Calculation

For a particular canal, the precision of the unsteady flow calculation, computer system software withstanding, is largely a function of three parameters; the time interval between hydraulic routing calculations, Δt, the length of reach separating the nodes used to define each reach, Δx, and the value of the finite difference weighting parameter, θ. (See Appendix A for a detailed description of these parameters.) To evaluate the relative significance of these parameters on the quality of the numerical solutions, a series of simple numerical experiments were performed as follows.

9.3.3.1 Variation in the Time Interval Between Calculations

In the first series of experiments, the following canal conditions were assumed:

1. three reaches, each 500 m long;
2. trapezoidal canal section, 3:1 side slopes, bottom width of 0.5 m;
3. longitudinal slope of 0.001;
4. hydraulic roughness, n = 0.02;
5. zero seepage losses;
6. flow of 0.1 m^3/s past the check structure of reach three;
7. twenty reach increments were used, (that is, 21 nodes); and,
8. the finite difference weighting parameter was set equal to 0.6 as suggested by Krishnappan (1981).

The flow at the inlet was instantaneously increased from 0.1 m^3/s to 0.5 m^3/s. The objective was to determine the maximum Δt at which no instability occurred and reasonable precision was achieved. The resulting surge was allowed to pass through the system and the simulation was terminated. Simulations were carried out for time intervals of 0.005, 0.05, and 0.2 hour. The discharge hydrographs at the canal inlet and at the end of each of the three reaches are plotted in Figure 9.3. It will be noted that as Δt increases, the curve tends to flatten and be displaced forward in time. Instabilities such as that exhibited by Reach #1 at Δt = 0.2 hour also occur.

The relative cost of these experiments were noted and are also plotted in Figure 9.3. Note that the cost of using a Δt = 0.05 hour is approximately one-fifth of the cost using a Δt = 0.005 hour and nearly the same cost as using a Δt = 0.2 hour.

A maximum value of Δt = 0.5 hour was chosen for use in future experiments on this canal. The loss in apparent analytical accuracy was considered to be adequately compensated for by the improved cost effectiveness of the computer simulations.

9.3.3.2 Variation in Reach Increment Length, Δx

In the second set of numerical experiments, the time interval had was set at 0.05 hours and a 2000 m reach examined. The 2000 m reach was chosen in order to examine the effect of a greater variety of values of Δx. Only one reach was examined since the first reach is subject to the most instability. The flow at the canal inlet was abruptly increased from 0.1 m^3/s to 0.2 m^3/s. The simulation was repeated for Δx equal to 100 m, 200m, and 400 m. The objective was to determine the maximum Δt at which no instability occurred and reasonable precision was achieved. The resulting discharge hydrographs are shown plotted in Figure 9.4. Note that the discharge hydrographs corresponding to Δx equal to 100 m and 200 m are approximately the same. For Δx = 400 m considerable numerical instability is exhibited. Considering that most of the cost saving was realized by using a Δx equal to 200 m and that the numerical instability was also removed, it was concluded that using a Δx equal to 200 m provided satisfactory precision in the unsteady hydraulic routing

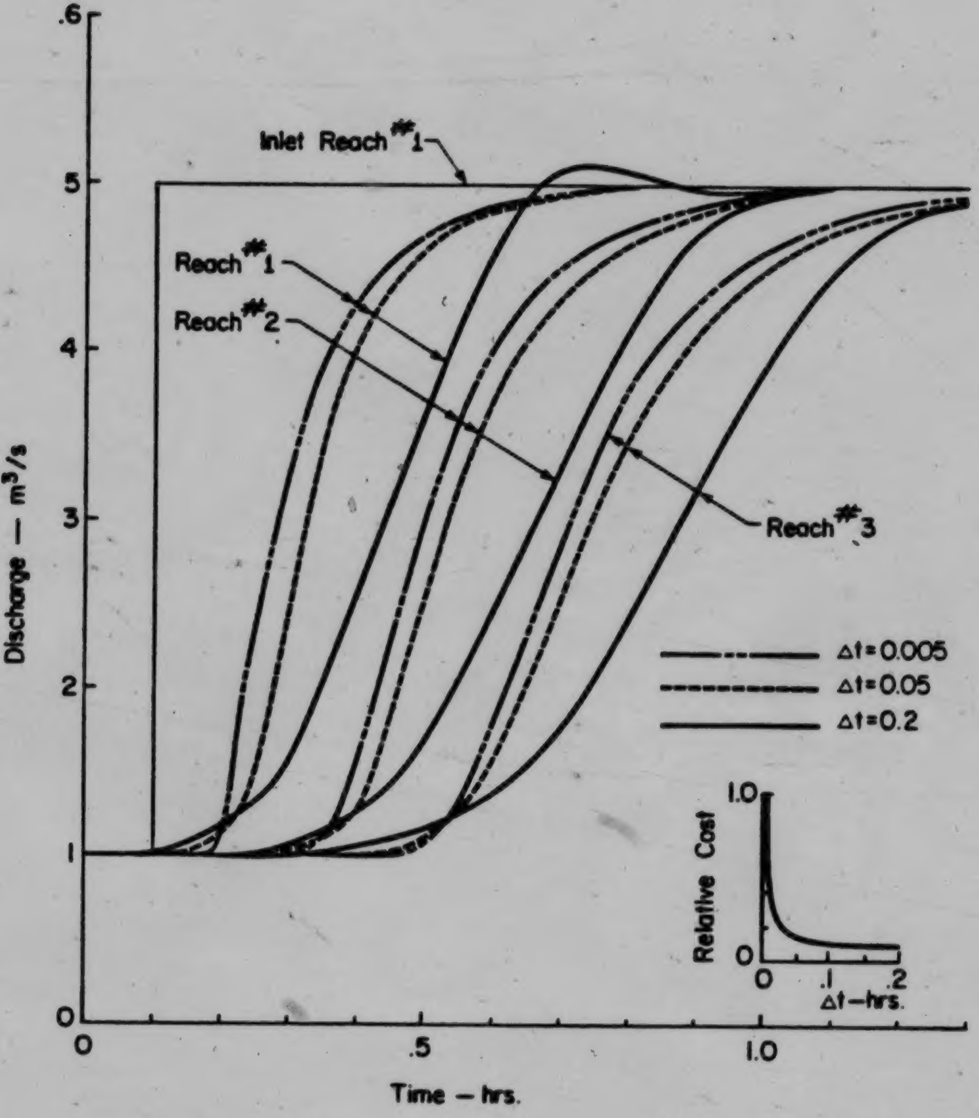


Figure 9.3 Variation in outlet hydrographs from reaches one, two, and three in response to a positive step change in the inlet hydrograph of the canal due to variations in the hydraulic routing time increment, Δt and relative cost of simulation versus hydraulic routing time increment.

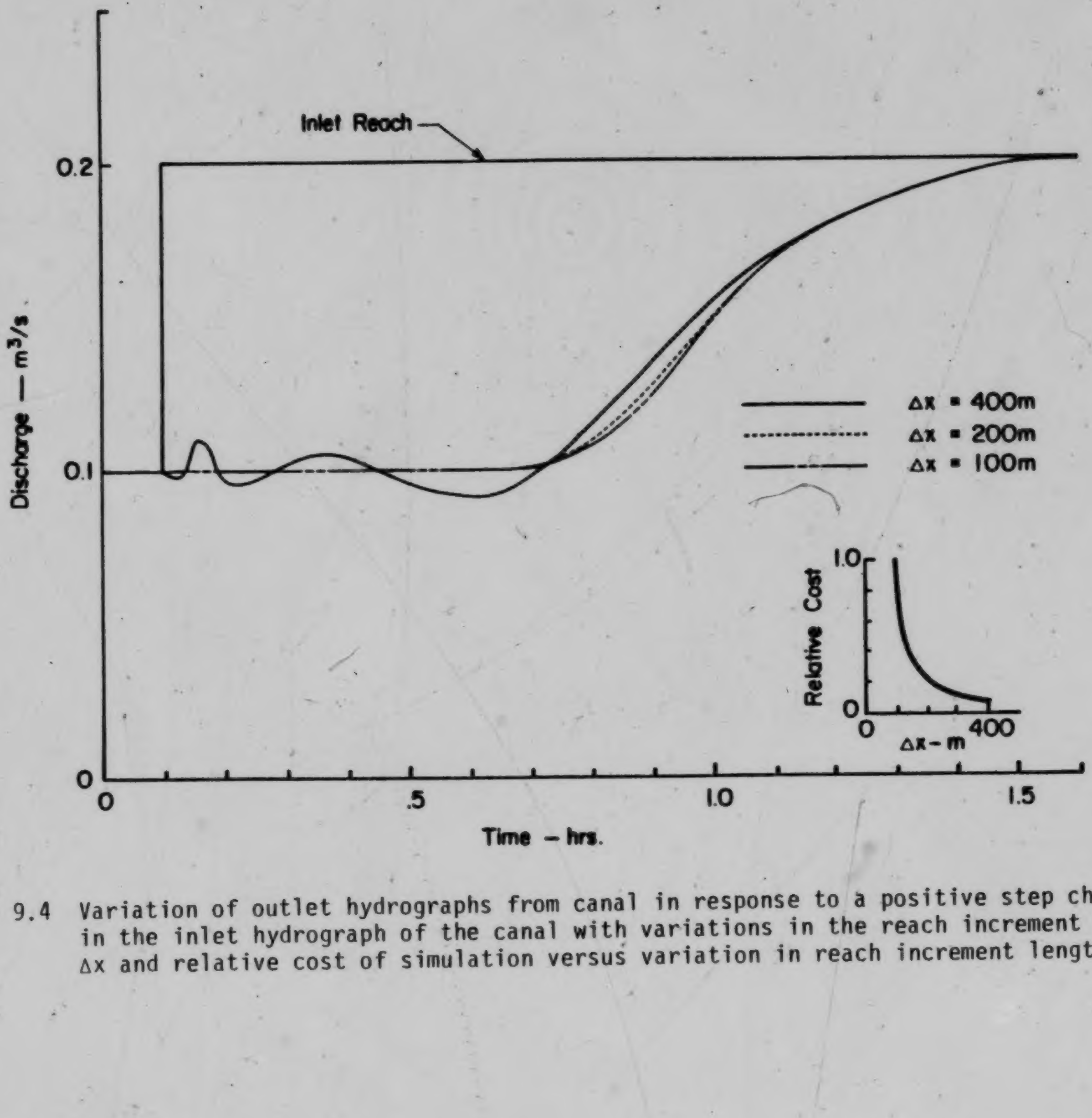


Figure 9.4 Variation of outlet hydrographs from canal in response to a positive step change in the inlet hydrograph of the canal with variations in the reach increment length, Δx and relative cost of simulation versus variation in reach increment length.

for the type of canal being examined.

9.3.3.3 Variation in the Finite Difference Weighting Parameter, θ

The third set of numerical experiments consisted of varying the finite difference weighting parameter θ for simulations in a 500 m reach. (At a $\theta = 0.5$, the numerical procedures contained in the unsteady flow routing subroutine is said to be fully centered and is theoretically stable. At θ greater than 0.5 the calculations are said to be unconditionally stable.) The time interval was set at 0.03 hours and Δx at 40 m. (twenty-one nodes). The discharge at the canal inlet was abruptly increased from 0.1 m^3/s to 0.5 m^3/s. The simulation was repeated for values of θ equal to 0.5, 0.55, and 0.8.

The resulting discharge hydrographs at the outlet of the reach are shown plotted in Figure 9.5. Note that a value of θ equal to 0.5 produces unstable results and is the least cost effective. Note as well that increasing θ results in a minor flattening of the output hydrograph. This effect cannot be considered a significant error in view of the error induced by varying Δt. The hydrograph generated when $\theta = 0.55$ approximates that of $\theta = 0.5$ but does not include the instabilities. Considering that the relative cost of simulation employing $\theta = 0.55$ is essentially equal to the cost of simulations employing $\theta = 0.80$ and that the discharge hydrograph produced by $\theta = 0.55$ is more precise, a value of θ equal to 0.55 was chosen for future numerical experiments on this canal.

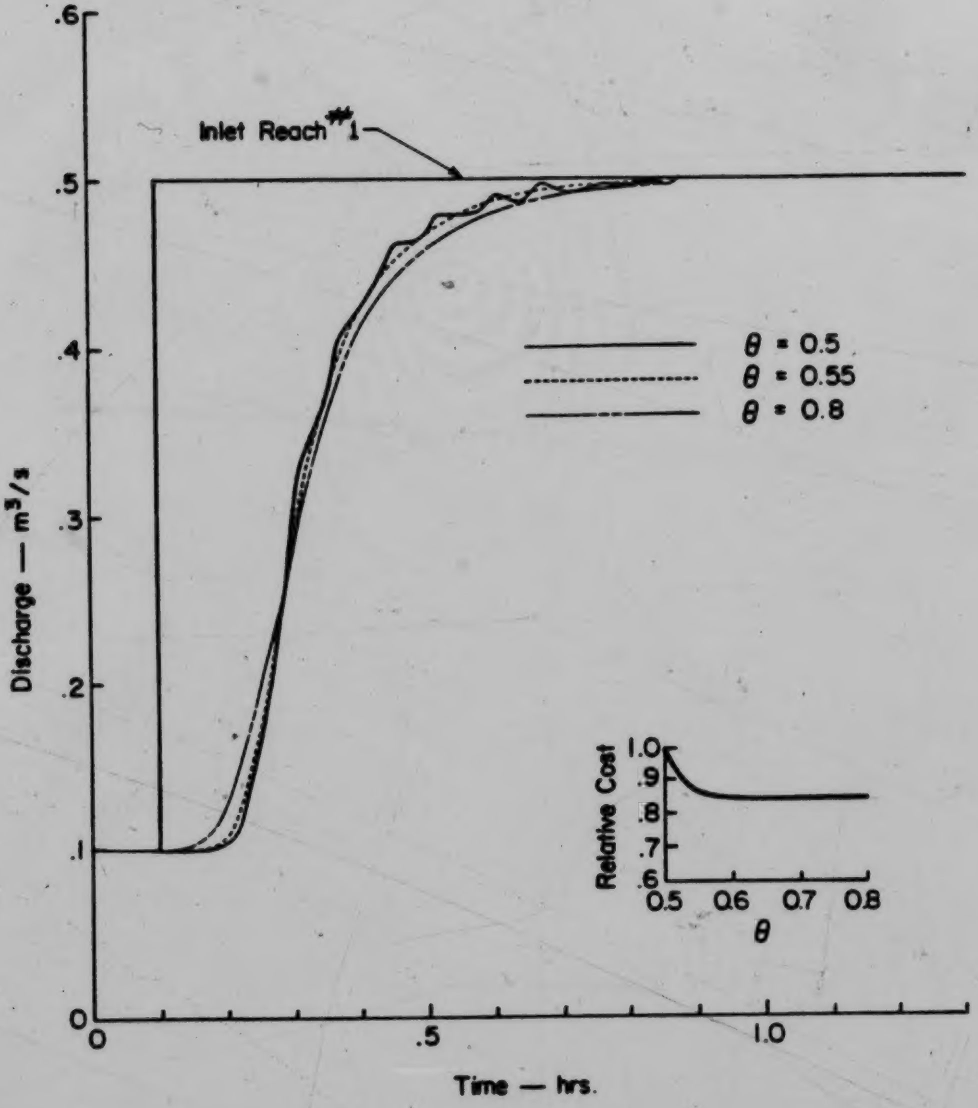


Figure 9.5 Variations in outlet hydrographs from canal in response to a positive step change in the inlet hydrograph of the canal with variations in the finite difference weighting parameter, θ and relative cost of simulation versus finite difference weighting parameter.

9.3.3.4 Other Forms of Sensitivity Analyses

Other more sophisticated methods for determining the sensitivity of the numerical solution to variation in Δt and Δx may be performed (those which employ the Courant Number which is equal to the product of the propogation velocity of the wave and the time increment divided by the length increment).

The effort required to employ these techniques is not usually justified. The concern in the simulation procedure is that the numerical solutions be stable and precise over the range of flow fluctuations the canal will experience. Instability is readily identified when it occurs and appropriate compensations in Δt and Δx made. Precision of the calculation is guaranteed over the entire range of flow variation if adequate precision is achieved during periods of maximum flow fluctuation.

9.4 Evaluation of the Potential Efficiency of Irrigation Conveyance Systems Operating in the Steady-State Flow Condition

The ICSS Model was applied to the study canal described in Section 9.2 to demonstrate how the maximum conveyance efficiency, as defined by Equation 7.15, would vary with changes in important canal physical characteristics which included reach length, seepage rate, roughness, discharge and side-slope. For these numerical experiments, it was not necessary to use the unsteady flow capabilities of the ICSS Model. Evaluations of the type performed would be virtually impossible to perform in the field.

9.4.1 Variation of Reach Length

Numerical experiments were performed for reach lengths of 200, 500, 1000, and 2000 metres. In all experiments, the following canal conditions were established:

- canal had a trapezoidal shape, bottom width of 0.5 metre and 3:1 side slopes;
- canal roughness, Manning's n = 0.02;
- seepage, Moritz Coefficient = 10;
- Δx = 200 m or less;
- Δt = 0.05 hr.;
- θ = 0.55;
- longitudinal slope = 0.001;
- check-structures of reaches one and two were set at minimum height;
- no water deliveries through farm turnouts in reaches one and two;
- farm water delivery of 0.1 m^3/s through turnout of reach number three;
- check-structure of reach number three was set at the minimum height to facilitate the water delivery through the farm turnout opened to its maximum; and,
- spill past the check-structure of reach number three was set at 0.001 m^3/s, (1 litre per second).

The results of these experiments are shown in Figure 9.6. It will be noted that $e_{c,max}$ approaches 100% as reach length tends to zero and

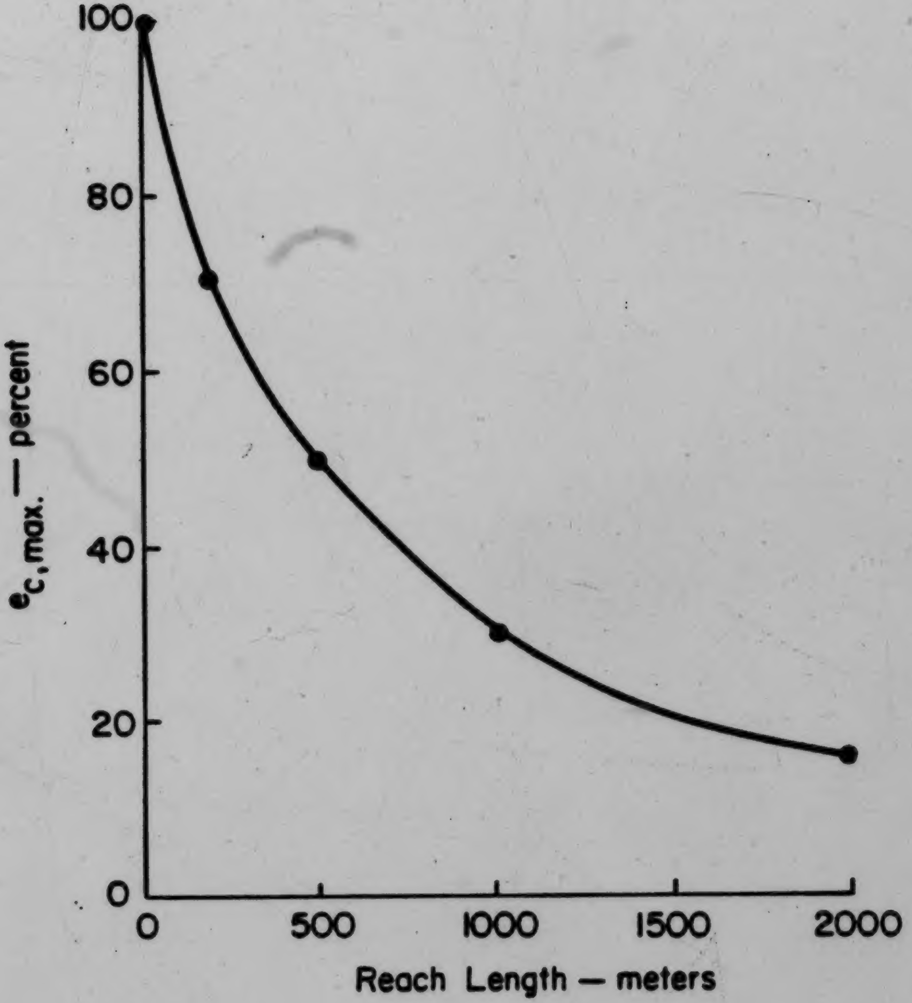


Figure 9.6 Maximum irrigation conveyance efficiency as a function of reach length.

that $e_{c,max}$ approaches 0% as reach length tends to infinity.

9.4.2 Variation in Seepage Rate

Numerical experiments were performed using several seepage rates as estimated by the Moritz Equation described in Appendix E. The maximum conveyance efficiency was evaluated for the Moritz Coefficients of 0, 1, 5, 10, 20, 30, and 40. Other canal conditions were similar to those described in Section 9.4.1 except that the reach lengths were fixed at 500 meters. The results of these experiments are shown in Figure 9.7. It will be noted that $e_{c,max}$ approaches 100% as the Moritz coefficient tends to zero and that $e_{c,max}$ approaches 0% as the Moritz coefficient tends to infinity.

9.4.3 Variation in Canal Roughness

Numerical experiments were performed using several canal hydraulic roughness conditions as described by Manning's n. The maximum conveyance efficiency was evaluated for values of Manning's n of 0.01, 0.02, 0.03, 0.05, 0.08, and 0.1. Other canal conditions were similar to those described in Section 9.4.1 except that the reach lengths were fixed at 500 m.

The results of these experiments are shown in Figure 9.8. It will be noted that there is an approximate straight line relationship between the value of n and $e_{c,max}$.

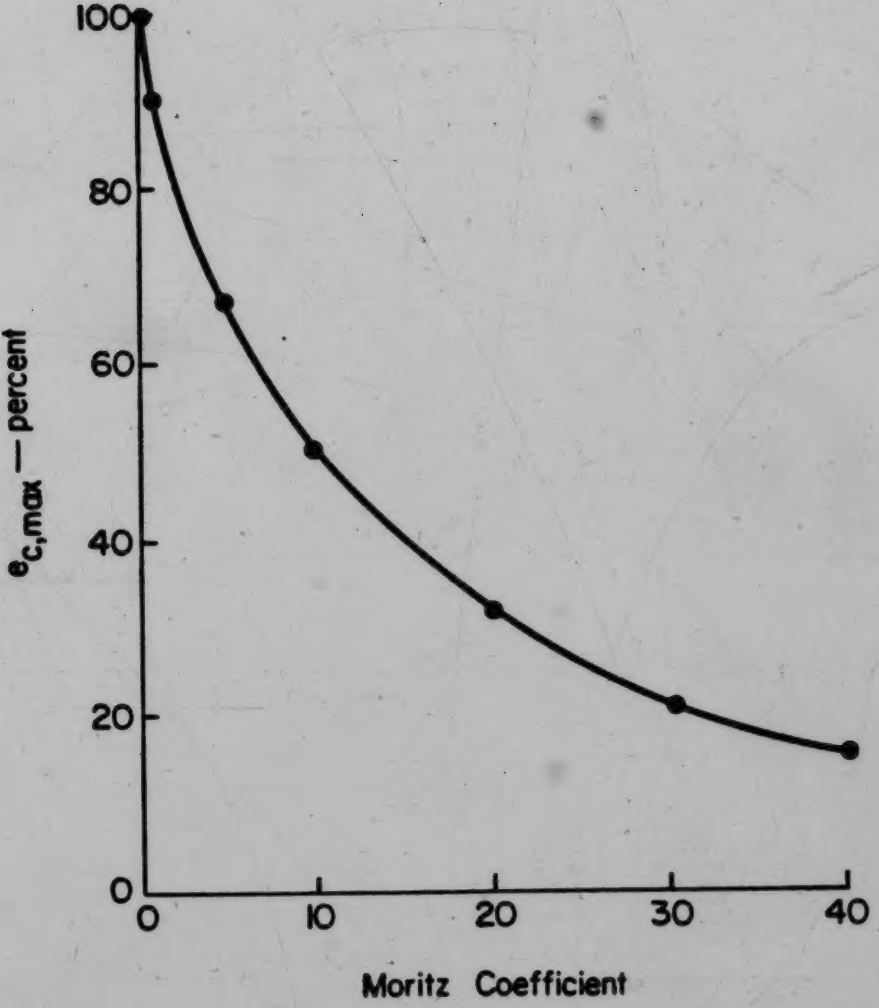


Figure 9.7 Maximum irrigation conveyance efficiency as a function of Moritz Coefficient.

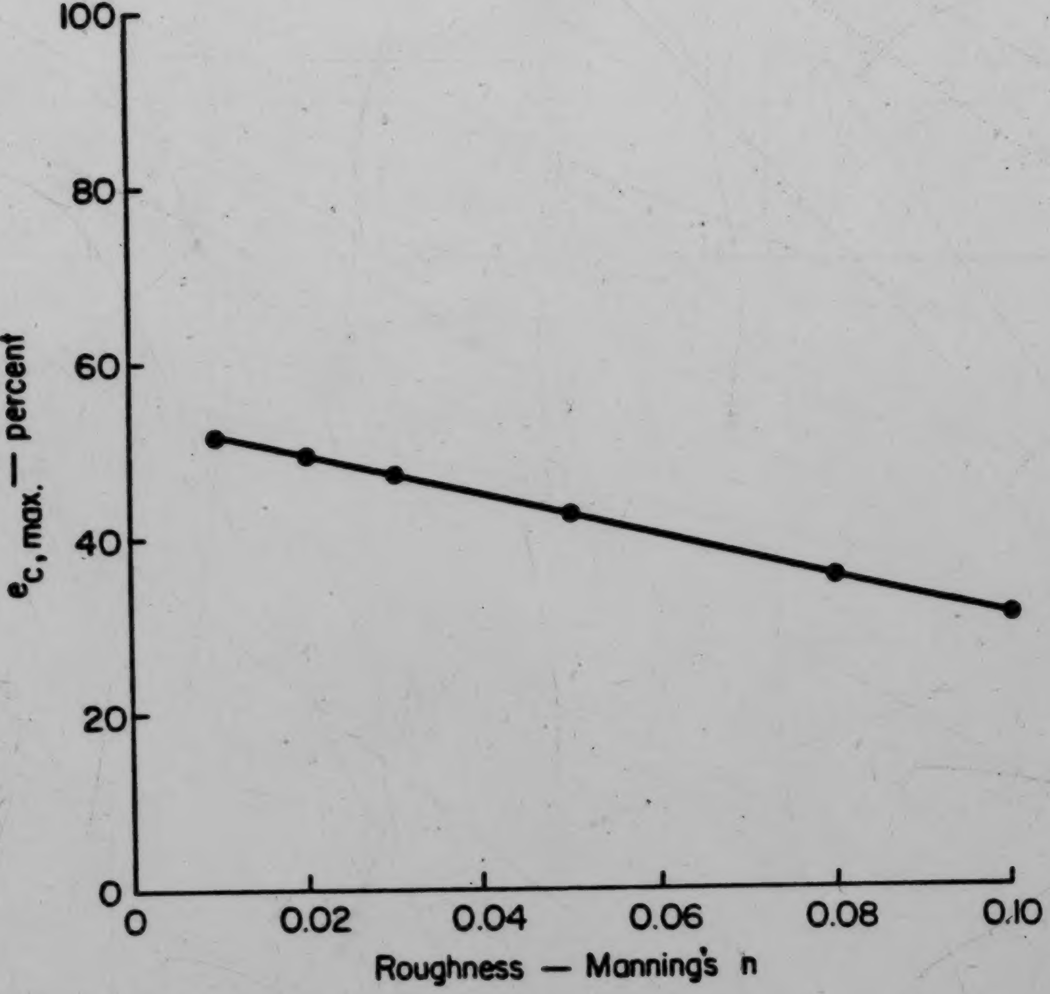


Figure 9.8 Maximum irrigation conveyance efficiency as a function channel roughness as described by Manning's n.

9.4.4 Variation in Discharge

Numerical experiments were performed using several discharge rates past the check-structure of reach number three. The farm water delivery at reach three was set to zero and the turnout was closed. The check-structure was set at minimum height. All of the water passing the check-structure of reach number three was assumed to be available for beneficial consumption. Other canal conditions were similar to those described in Section 9.5.1 except that the reach lengths were fixed at 500 metres. Maximum conveyance efficiency was determined for flows past the check-structure equal to 0.001, 0.1, 0.2, 0.4, 0.5, and 0.8 m^3/s.

The results of these experiments are shown in Figure 9.9. It will be noted that $e_{c,max}$ approaches zero as the discharge at the reach outlet tends to zero and that $e_{c,max}$ approaches 100% as discharge at the outlet tends to infinity.

9.4.5 Variation in Canal side Slope

Numerical experiments were performed using three values of canal side slope; 2:1, 3:1, and 4:1. Other canal conditions were similar to those described in Section 9.4.1 except that the reach lengths were fixed at 500 m. No significant variation in $e_{c,max}$, was indicated over the range in side slope examined. It is expected that a more pronounced variation of $e_{c,max}$ would result with variation in the bottom width of the canal.

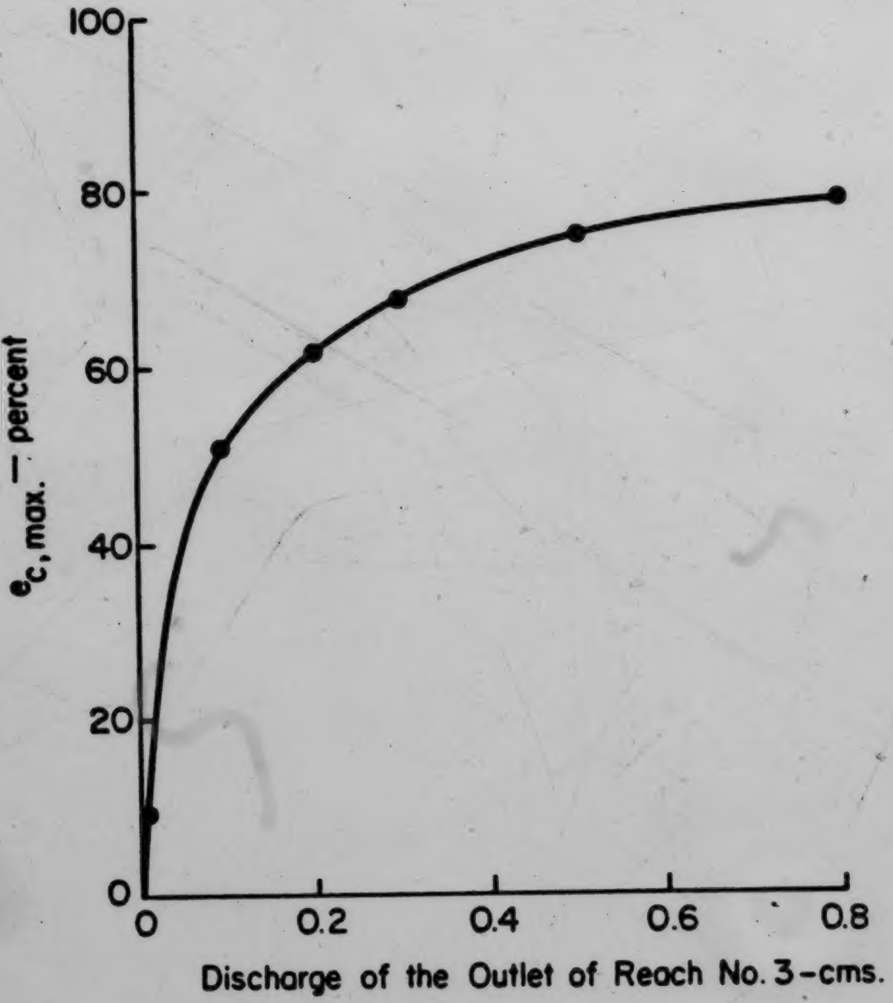


Figure 9.9 Maximum irrigation conveyance system efficiency as a function of the discharge of water through canal.

9.5 Evaluation of the Efficiency of Irrigation Conveyance System Operational Procedures

The ICSS Model was applied to the study canal described in Section 9.2 to demonstrate how the conveyance, physical conveyance and operational efficiency concepts could be used to evaluate specific operational procedures. For this numerical experiment extensive use was made of the unsteady flow segment of the ICSS Model.

9.5.1 Description of Study Canal

The following study conditions were employed in the simulations:

- canal was trapezoidal in shape, bottom width = 0.5 m and 3:1 side slopes;
- canal roughness, Manning's n = 0.02;
- seepage, Moritz Coefficient = 0.10;
- reach length = 500 m;
- Δx = 100 m;
- hydraulic Δt = 0.05 hr.;
- hydrologic Δt = 0.05 hr.;
- θ = 0.55;
- longitudinal slope = 0.001;
- check-structures in all reaches are set at minimum height;
- farm turnouts in all reaches were closed;
- flow passing the check-structure of reach number three was 0.1 m^3/s and was assumed to be available for beneficial consumption;

- all depth and discharge observation errors set equal to zero;
- all structure adjustment errors set equal to zero;
- the upper and lower limits to farm water delivery were set at ±10% of the farm water requirements;
- the upper and lower limits to the water delivery downstream of the check-structure in reach three were set at ±10% of the water delivery requirements; and,
- the check-structure minimum adjustment increment was set at 0.001 m (in order to permit calculation of $e_{c,max}$).

9.5.2 Operational Procedure Examined

The objective of the canal operational procedure examined was to deliver 0.1 m^3/s of water through the farm turnout of reach number three while maintaining the 0.1 m^3/s flow past the check-structure of reach number three. The operational procedure examined consisted of the following steps:

1. Increase the flow rate at the canal inlet sufficient to deliver water through the turnout.
2. Immediately raise the height of the check-structure at reach number three in order to permit delivery of water through turnout.
3. Wait until steady-state conditions are approximated.
4. Operate the farm turnout and begin the delivery.
5. Allow steady-state conditions to be achieved.
6. Terminate delivery, return check-structure to original height and return flow rate at inlet to original value.

9.5.3 Numerical Experiment

The numerical experiment consists of two parts which may be called the steady-state and unsteady-state procedures.

The steady-state procedure is applied first. It consists of determining the minimum check height that would be required to deliver the 0.1 m^3/s of water to the farmer at the maximum turnout opening while maintaining the 0.1 m^3/s flow past the check. The ICSS Model was simply allowed to initialize itself and then the simulation was terminated. As well as providing the information required to evaluate $e_{c,max}$ and later, e_o, the procedure also determined the minimum flow rate at the canal inlet and the minimum check height in reach three required to make the delivery to the farmer.

The unsteady procedure required that the study conditions be returned to those indicated in Section 9.6.1. The execution of the program was started and the operation procedure as described in Section 9.6.2 was performed interactively with the following additions: after step 7 was completed, the system was allowed to achieve steady-state (a condition that could be verified at the canal operator's discretion), after which the simulation was terminated. The flow rate and check height adjustments were purposely exaggerated beyond those required to demonstrate the model's capabilities.

9.5.4 Results

The results of the numerical experiments are illustrated in Figure 9.10 (a), (b) and (c). Note that the farm delivery started two hours

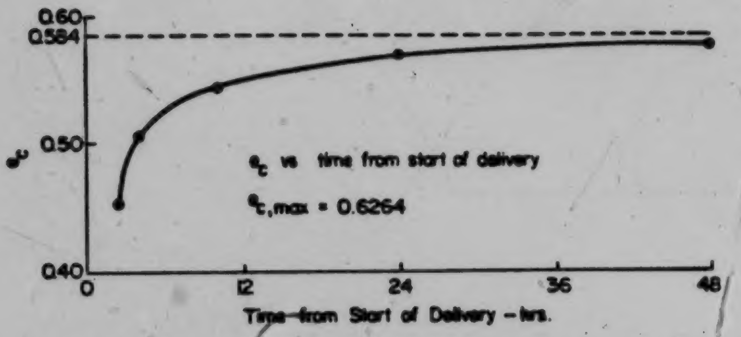


Figure 9.10(a) Conveyance efficiency as a function of time from start of water delivery.

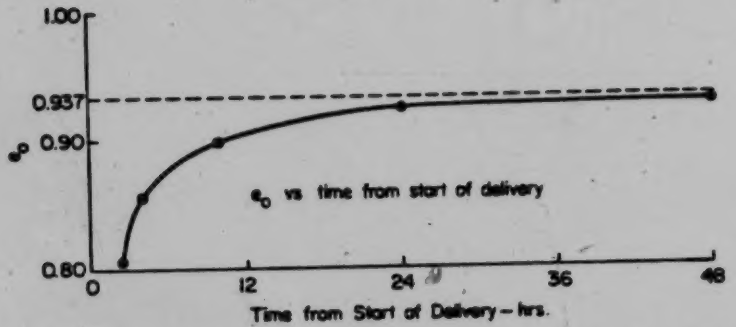


Figure 9.10(b) Operational conveyance efficiency as a function of time from start of water delivery.

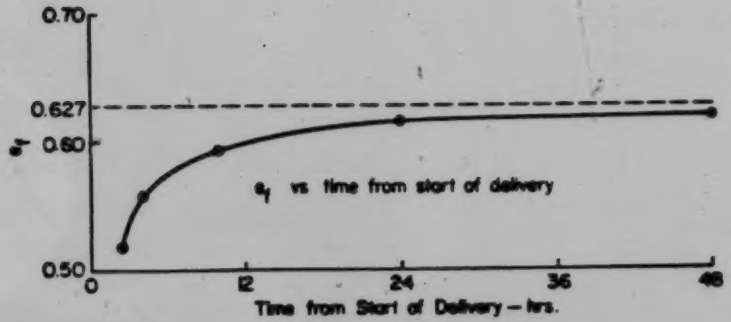


Figure 9.10(c) Physical conveyance efficiency as a function of time from start of water delivery.

after water was released into the canal. The efficiencies indicated by the dashed lines are the maximum efficiencies that this particular water delivery achieved using the operational procedures described. The shape of the curves indicate that the maximum efficiencies are approached assymptotically as the relative magnitude of initial changes in storage, seepage loss and spillage lose their significance. The maximum conveyance efficiency possible is 0.624 as compared to the maximum conveyance efficiency that will be achieved, 0.584. Note that the shorter the total duration of the water deliveries, the lower the achievable conveyance, operating and physical conveyance efficiency.

9.6 Discussion

It is apparent that the ICSS Model has sufficient flexibility that it can be used to evaluate the impact on the efficiency of a considerable variety of irrigation canal physical conditions and operating procedures. Successful application of the model, however, requires careful attention to the specific characteristics of the conveyance system being simulated.

The algorithms used to simulate the steady and unsteady flow conditions cannot be applied carelessly. If extreme hydraulic conditions are to be examined, instabilities in the numerical techniques may result which would either render the results useless or prohibit the calculation at all. This is particularly true of the unsteady flow algorithm. To ensure that analytically precise results are achieved, an analysis similar to that discussed in Section 9.4 should be performed over the entire range of flow conditions the future numerical experiments will examine. By following this procedure, the user can be reasonably assured that accurate results are being obtained at minimum cost.

The ICSS Model can generate a formidable volume of data if all reporting options are used. Cost savings may be realized by suppressing much of this output in production runs when much of the potential output is not required.

Finally, application of the ICSS Model requires considerable planning. The user must be thoroughly aware of all important physical and operational details of the conveyance system he is intending to examine. This is necessary to ensure that the algorithms he develops to simulate the groups of hydraulic structures describe their physical and operating capability in a "realistic" manner and to ensure that numerical experiments for which the algorithms do not apply are not being attempted. The application of the ICSS Model, at least for purposes of determining the efficiency characteristics of a particular irrigation conveyance system, requires the same planning as that needed to design an observation program intended to monitor a similar conveyance system in the field. Once correctly configured, the advantages of using the ICSS Model are that a considerable number of physical and operational variations may be examined in detail in the same period a field observation program would require to simply assemble the necessary apparatus.

CHAPTER 10

SUMMARY AND CONCLUSIONS

Procedures and techniques with which to describe all variations of irrigation conveyance systems and evaluate their performance have been developed and demonstrated. The systems approach, defined in the most general sense, has been successfully applied. All of the research objectives stated in Chapter Two have been satisfied. That is:

1. The water demands irrigation conveyance systems are intended to satisfy have been identified and a general terminology has been developed to describe them.
2. The physical components which comprise irrigation conveyance systems have been identified and a general classification system has been constructed with which the component function within the irrigation conveyance system is clearly described.
3. The subsystems of physical components which comprise irrigation conveyance systems have been identified and described in such a manner that conveyance system configuration may be clearly described.
4. All general techniques used to operate irrigation conveyance systems and subsystems have been identified and described.
5. A performance criteria with which to evaluate the performance and efficiency of irrigation conveyance systems, subsystems, and components has been developed.

6. A general computer model, which enables the evaluation of irrigation conveyance system performance, has been developed and demonstrated.

A comprehensive terminology for describing farm water demands was developed on the basis of an analysis of the works of Repogle and Merriam (1980) and Sagardoy et al. (1982). The terminology developed describes farm water demand in a manner meaningful for the discussion of irrigation conveyance system design, operation, and performance evaluation.

A system for classification of irrigation conveyance system components was developed which provides a method for the identification, description and comparison of the numerous and diverse components comprising irrigation conveyance systems. The classification system is comprehensive in that all structural components found within irrigation conveyance systems may be considered. Use of the classification system permits the concise description of component and system operations as well as suggesting the criteria with which they might be evaluated. Use of the classification system facilitates the conceptualization and development of mathematical models of conveyance systems and subsystems by providing a guide with which investigators can recognize individual component functions and recognize their significant hydraulic and operational characteristics.

A terminology for describing irrigation conveyance systems and subsystems was developed. The terminology was modelled after the one introduced by the ICID (1978), however, significant modifications were made to their work in order to achieve a greater degree of comprehensiveness and remove ambiguity. All components which comprise irrigation conveyance systems are described with respect to the water demands they are intended to satisfy. Within individual conveyance systems, similarly classified components could be expected to exhibit a degree of physical similarity. By indicating the relationship of components to the water demand being satisfied, the classification system provides an approximate index of operation complexity. Use of the classification system provides a rational means with which to identify similar subsystems of conveyance systems for purposes of comparative performance evaluation studies. Combined with the system for classifying irrigation conveyance system components, the subsystem classification system permits concise relevant discussion of irrigation conveyance system design and operation.

All of the general techniques used to operate irrigation conveyance systems and subsystems have been identified and described. The description included orientation and type of control. Emphasis was placed on demand-oriented conveyance systems. Manual and automatic upstream- and downstream-controlled systems and systems operating using dynamic regulation techniques were discussed.

A criteria was developed with which to evaluate the performance and efficiency of specific irrigation conveyance systems, subsystems, and components. Methods were introduced with which to evaluate conveyance systems, maximum conveyance systems, operational and physical conveyance efficiency of irrigation conveyance systems, and the performance of several individual components.

A computer model was developed and demonstrated which enables the evaluation of irrigation conveyance system performance. The model, named the Irrigation Conveyance System Simulation Model or ICSS Model is applicable to a limited but well defined selection of irrigation components, systems and methods of operation. Despite these limitations provisions were made in the structure of the ICSS Model to permit almost unlimited expansion, extension and revision in the hope that the model's utility could be maintained over the broadest range of applications.

The ICSS Model was demonstrated by considering a distributary canal typical of Southern Alberta, Canada. Analyses were performed with which the analytical integrity of the ICSS Model, in the particular application chosen, was demonstrated. As well, the ICSS Model was shown to possess sufficient flexibility to evaluate the impact on the efficiency of a variety of conveyance system characteristics and operation procedures.

CHAPTER 11

DISCUSSION

The systems analysis of irrigation conveyance systems provides the context within which to consider previously performed research on irrigation conveyance systems and permits the identification and evaluation of the potential contribution of future research endeavors. Specific areas of research which need to be pursued include at least the following:

1. development of a field observation program with which to calibrate/validate specific ICSS Model applications;
2. development of rating curves for each of the myriad of hydraulic structures comprising irrigation conveyance systems;
3. extension the ICSS Model to permit the simulation of canal priming and dewatering processes;
4. extension of the ICSS Model to permit the simulation of a greater variety of hydrological processes;
5. extension of the ICSS Model to permit the consideration of the full selection of hydraulic structures identified in Chapter 4;
6. extension of the ICSS Model to permit the simulation of multi-canal conveyance systems and subsystems;
7. development of improved algorithms for consideration of seepage processes within the ICSS Model;
8. development of algorithms for consideration of variable surface roughness phenomena in the ICSS Model;

9. development of algorithms for consideration of wind effects in the ICSS Model;
10. development of procedures to determine optimum operational techniques, manual or automatic, for particular conveyance systems and subsystems;
11. development of field techniques with which to evaluate conveyance, operation and physical conveyance efficiency in a timely manner; and,
12. the performance of numerous sensitivity analyses with which to examine the impact on irrigation conveyance system performance of the numerous variables describing the physical and operational characteristics of conveyance systems.

The opportunity and need for relevant applied research on irrigation conveyance system design, operation and maintenance appear, at least at the time of writing, limitless. This is perhaps why the Task Committee on the Status of Irrigation and Drainage Research of the Committee on Research of the Irrigation and Drainage Division of the American Society for Civil Engineers, ASCE (1984), identified as one of their four high priority research objectives; "to develop improved systems and practices to store and deliver water supplies to farms as needed for efficient crop production". The Task Committee identified two approaches by which this objective may be satisfied. They are to:

1. "develop improved water storage, control and distribution systems, and (to) develop improved institutional systems, management policies and practices to provide water as needed

for crop production and to minimize evaporation, non-recoverable seepage losses and operational waste;" and,

2. "develop improved and cost-effective materials and equipment to improve water management, including water measurement and reduce non-recoverable on farm water losses".

It is believed that the systems analysis of irrigation conveyance systems as defined and described in the previous discussion has at least brought the satisfaction of this objective one step closer.

CHAPTER 12

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APPENDIX A

IMPLICIT FINITE DIFFERENCE SOLUTION OF THE ONE DIMENSIONAL EQUATIONS OF UNSTEADY GRADUALLY VARYING OPEN CHANNEL FLOW

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IMPLICIT FINITE DIFFERENCE SOLUTION OF THE ONE DIMENSIONAL EQUATIONS OF UNSTEADY GRADUALLY VARYING OPEN CHANNEL FLOW

A.1 Derivation of One-Dimensional Equations

The one-dimensional equations of unsteady gradually varying open-channel flow, also known as the St. Venant Equations, have been rigourously derived and examined by many investigators including; Stoker (1957), Chow (1959), Henderson (1966), Strelkoff (1969), Eagleson (1970), Yevjevich and Barnes (1970), Yen (1972) and (1973) and Krishnappan (1981). The derivation presented here is essentially a restatement of their work.

A.1.1 Continuity Equation

Considering the channel element shown in Figure A.1, the continuity equation, (which is a statement of the law of conservation of mass), may be given by the relation:

$$\begin{array}{c}\text{Volume of}\\ \text{Water Entering}\\ \text{the Channel}\\ \text{Element}\end{array} - \begin{array}{c}\text{Volume of}\\ \text{Water Leaving}\\ \text{the Channel}\\ \text{Element}\end{array} = \begin{array}{c}\text{Rate of Change}\\ \text{of Water Storage}\\ \text{in Channel}\\ \text{Element}\end{array} \tag{A.1}$$

The volume of water entering the channel element is given by the relation:

$$\begin{array}{l}\text{Volume of Water}\\ \text{Entering the}\\ \text{Channel Element}\end{array} = \{[A - \frac{\partial A}{\partial x}\frac{\Delta x}{2}][V - \frac{\partial V}{\partial x}\frac{\Delta x}{2}] + q_i\Delta x + p\Delta x\} \tag{A.2}$$

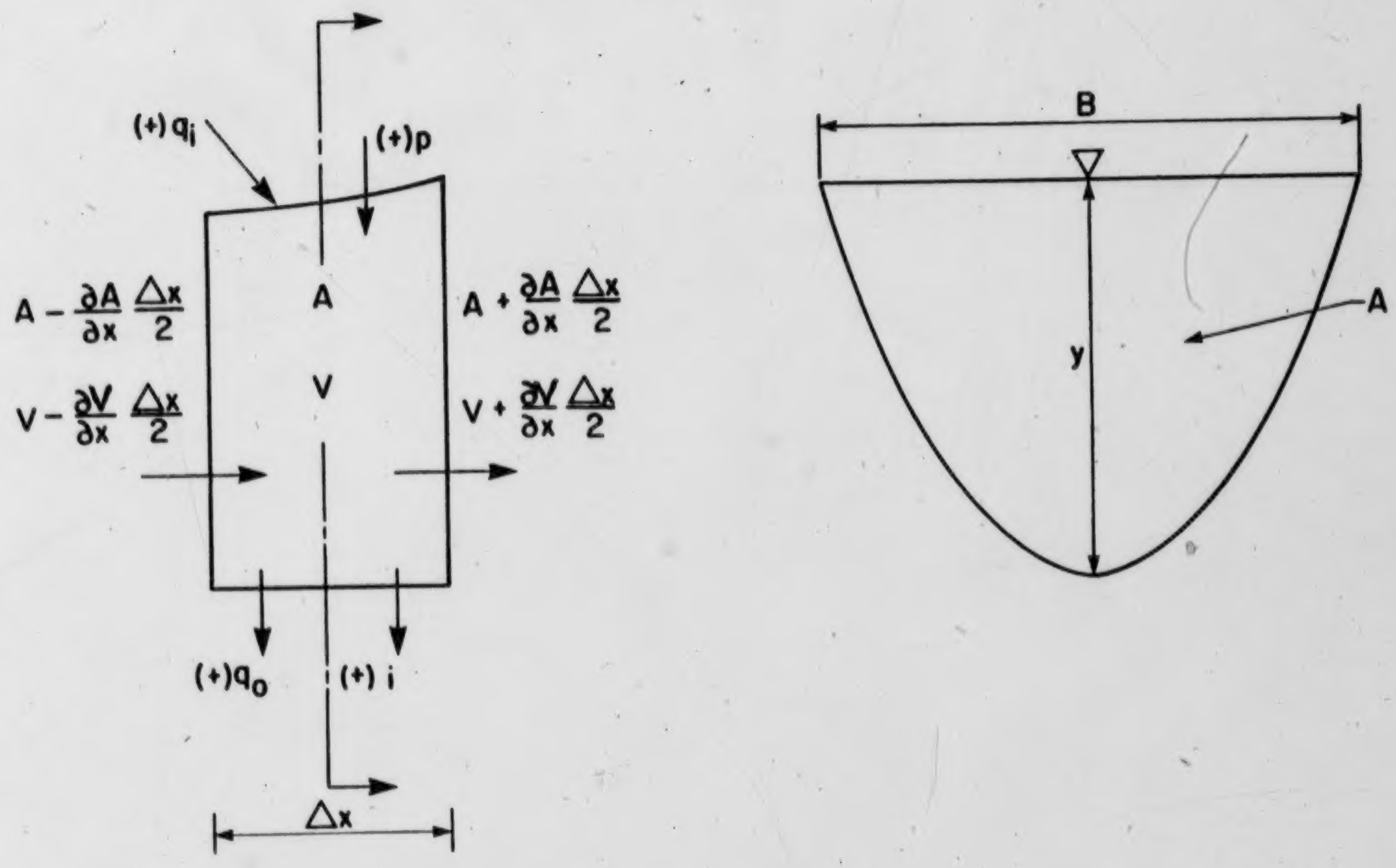

(+) q_i
(+)p
$A - \frac{\partial A}{\partial x}\frac{\Delta x}{2}$
A
$A + \frac{\partial A}{\partial x}\frac{\Delta x}{2}$
$V - \frac{\partial V}{\partial x}\frac{\Delta x}{2}$
V
$V + \frac{\partial V}{\partial x}\frac{\Delta x}{2}$
(+)q_o
(+) i
Δx
B
y
A

Figure A.1 Definition sketch unit channel element used for derivation of the continuity equation.

where: A = the cross-sectional area at the centre of the channel element.

V = the average velocity of the water at the centre of the channel element.

Δx = length of channel element.

q_i = rate of bulk lateral inflow per unit length of channel. Bulk lateral in flow enters the channel with a velocity component, V_i, in the direction of channel flow.

p = rate of distributed lateral inflow per unit length at channel. Distributed lateral inflow enters the channel with no velocity component in the direction of channel flow.

The volume of water leaving the channel element is given by the relation:

$$\begin{array}{l}\text{Volume of Water}\\ \text{Leaving the}\\ \text{Channel Element}\end{array} = \{[A + \frac{\partial A}{\partial x}\frac{\Delta x}{2}][V + \frac{\partial V}{\partial x}\frac{\Delta x}{2}] + q_o\Delta x + i\Delta x\} \qquad (A.3)$$

where: q_o = rate of bulk lateral outflow per unit length of channel. Bulk lateral outflow leaves the channel with the average velocity, V, of the water in the channel.

i = rate of distributed water outflow per unit length of channel. Distributed water outflow leaves the channel with no velocity component in the direction of channel flow.

The rate of change of storage in the channel element is given by the relation:

$$\begin{array}{l}\text{rate of change of}\\ \text{storage in}\\ \text{channel element}\end{array} = \frac{\partial A}{\partial t}\Delta x \qquad (A.4)$$

Recognizing that the net rate of flow in the channel element is equal to the rate of change of water storage in the channel, Equations (A.2), (A.3) and (A.4) may be combined as follows:

$$\{[A - \frac{\partial A}{\partial x}\frac{\Delta x}{2}][V - \frac{\partial V}{\partial x}\frac{\Delta x}{2}] + q_i\Delta x + p\Delta x\} - \{[A + \frac{\partial A}{\partial x}\frac{\Delta x}{2}][V + \frac{\partial V}{\partial x}\frac{\Delta x}{2}]$$

$$+ q_o\Delta x + i\Delta x\} = \frac{\partial A}{\partial t}\Delta x \qquad (A.5)$$

which reduces to the following form of the continuity equation:

$$\frac{A\partial V}{\partial x} + \frac{V\partial A}{\partial x} + \frac{\partial A}{\partial t} - (q_i - q_o + p - i) = 0 \qquad (A.6)$$

Several other forms of the continuity equation may be derived. If it is noted that,

$$\frac{A\partial V}{\partial x} + \frac{V\partial A}{\partial x} = \frac{\partial(VA)}{\partial x} \qquad (A.7)$$

Equation (5) may be written as

$$\frac{\partial(VA)}{\partial x} + \frac{\partial(A)}{\partial t} - (q_i - q_o + p - i) = 0 \qquad (A.8)$$

or as

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - (q_i - q_o + p - i) = 0 \qquad (A.9)$$

where Q is equal to the discharge through the centre of the channel element.

Since the channel is considered non-prismatic; that is, the cross-sectional area of the channel will vary with location, x, along the channel for a specific depth y, the term ∂A/dx may be written as:

$$\frac{\partial A}{\partial x} = \frac{\partial A}{\partial y} \cdot \frac{\partial y}{\partial x} + A_x^y \qquad (A.10)$$

where A_x^y is the rate of change of cross-sectional area with respect to x with depth y of the channel held constant. Recognizing that

$$\frac{\partial A}{\partial y} = B\,\frac{\partial y}{\partial y} = B \qquad (A.11)$$

the width of water surface at the centre of the channel element, Equation (A.10) becomes

$$\frac{\partial A}{\partial y} = B\,\frac{\partial y}{\partial x} + A_x^y \qquad (A.12)$$

Substituting Equation (A.12) into Equation (A.6) and noting that

$$\frac{\partial A}{\partial t} = B\,\frac{\partial y}{\partial t} \qquad (A.13)$$

another common form of the continuity equation results:

$$A\,\frac{\partial V}{\partial x} + VB\,\frac{\partial y}{\partial x} + B\,\frac{\partial y}{\partial t} + VA_x^y - (q_i - q_o + p - i) = 0 \qquad (A.14)$$

If the channel is considered prismatic; that is, $A_x^y = 0$ and Equation (A.14) is divided by B, the following useful form of the continuity equation results:

$$\frac{A}{B}\,\frac{\partial V}{\partial x} + V\,\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} - \frac{1}{B}\,(q_i - q_o + p - i) = 0 \qquad (A.15)$$

If the channel is considered to have a wide rectangular cross-section as well, only a unit width of the channel section may be considered, that is, $B = 1$ and $A = y \cdot 1$. Equation (A.15) becomes:

$$y \frac{\partial V}{\partial x} + V \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} - (q_i - q_o + p - i) = 0 \qquad (A.16)$$

or as;

$$\frac{\partial (Vy)}{\partial x} + \frac{\partial y}{\partial t} - (q_i - q_o + p - i) = 0 \qquad (A.17)$$

A.1.2 Momentum Equation

The momentum equation, (Newton's second law), may be stated as follows:

$$\begin{array}{c}\text{Net Rate of}\\ \text{Momentum Entering}\\ \text{the Channel Element}\end{array} + \begin{array}{c}\text{Sum of Forces}\\ \text{Acting on the}\\ \text{Channel Element}\end{array} = \begin{array}{c}\text{Rate of}\\ \text{Accumulation}\\ \text{of Momentum in}\\ \text{Channel Element}\end{array} \qquad (A.18)$$

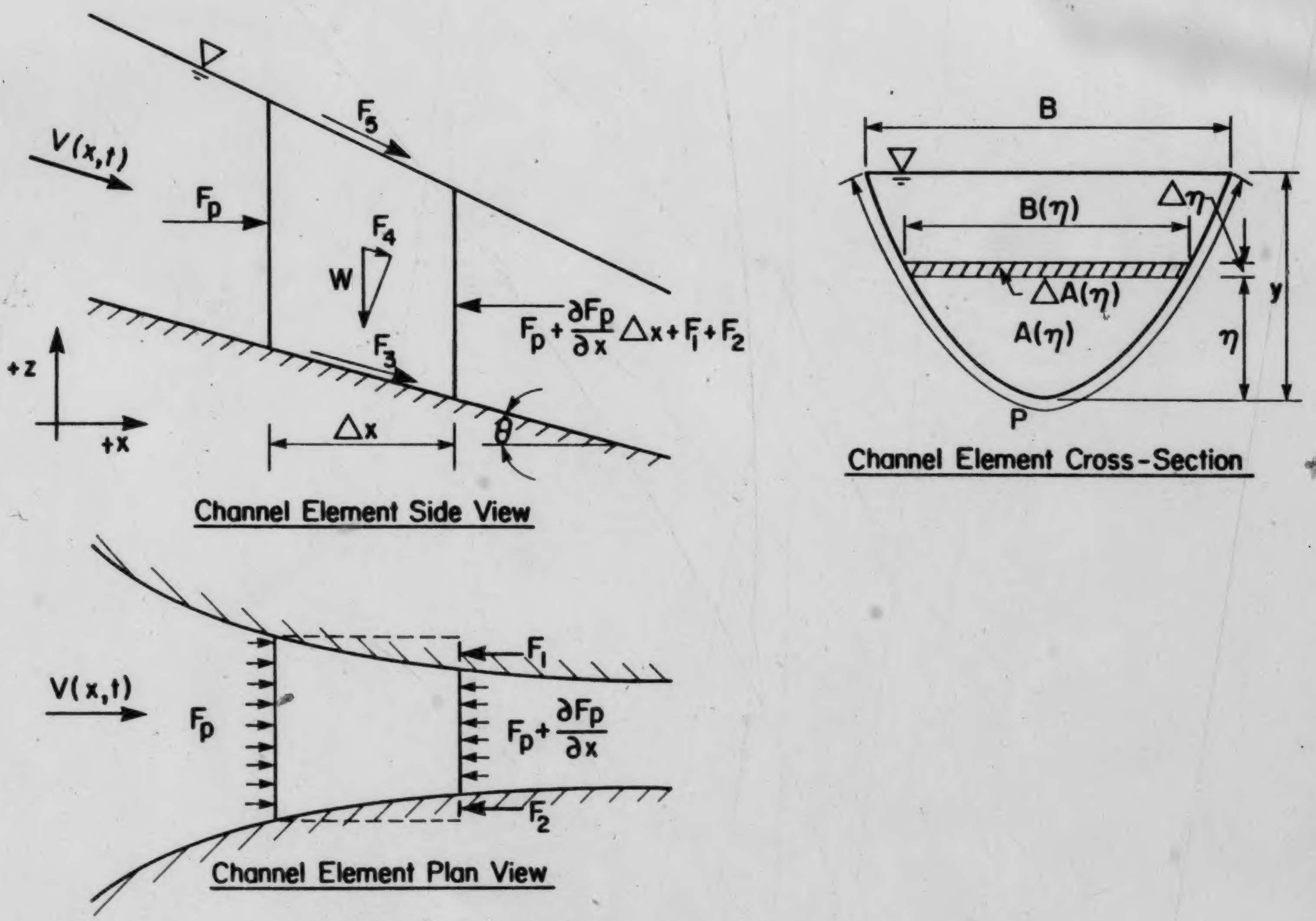
If it is assumed that the velocity distribution across the channel section is uniform; that is, the velocity distribution coefficients as given by the relations

$$\alpha = \frac{1}{AV^3} \iint_A V^3 \, \partial A = 1 \qquad (A.19)$$

and

$$\beta = \frac{1}{AV^2} \iint_A V^2 \, \partial A = 1 \qquad (A.20)$$

the rate of momentum entering the channel element, considering Figure A.1, is given by the relation:


V(x,t)
F_5
F_p
F_4
W
F_3
$F_p + \frac{\partial F_p}{\partial x} \Delta x + F_1 + F_2$
+z
+x
Δx
θ
B
$B(\eta)$
$\Delta\eta$
$\Delta A(\eta)$
$A(\eta)$
y
η
P
V(x,t)
F_p
F_1
$F_p + \frac{\partial F_p}{\partial x}$
F_2

Channel Element Side View

Channel Element Cross-Section

Channel Element Plan View

Figure A.2 Definition sketch used for derivation of the momentum equation.

$$\begin{array}{l}\text{Rate of Momentum} \\ \text{Entering the} \\ \text{Channel Element}\end{array} = \rho\{V(VA) - \frac{\partial(V(VA))}{\partial x}\frac{\Delta x}{2}\} +$$

$$+ \rho q_i \Delta x(V_i) + \rho p \Delta x \cdot (0) \qquad (A.21)$$

where: ρ = density of water; and,

V_i = velocity component in direction of channel flow of bulk lateral inflow

Note that the distributed lateral inflow, p, has no velocity component in the direction of channel flow.

The rate of momentum leaving the channel element is given by the relation:

$$\begin{array}{l}\text{Rate of Momentum} \\ \text{Leaving the} \\ \text{Channel Element}\end{array} = \rho\{V(VA) + \frac{\partial(V(VA))}{\partial x}\frac{\Delta x}{2}\} +$$

$$+ \rho q_o \Delta x(V) + \rho i \Delta x(0) \qquad (A.22)$$

Note that it is assumed that bulk lateral outflow, q_o, leaves the channel with the same velocity of the water in the channel and that the distributed lateral outflow, i, leaves the channel with no velocity component into the direction of water flow in the channel.

The net rate of momentum entering the channel element is simply the difference between Equations (A.21) and (A.22). That is;

$$\begin{array}{l}\text{Net Rate} \\ \text{of Momentum} \\ \text{Entering the} \\ \text{Channel Element}\end{array} = - \rho\Delta x \frac{\partial(V(VA))}{\partial x} + \rho q_i \Delta x V_i - \rho q_o \Delta x V \qquad (A.23)$$

The sum of forces acting on the channel element, (see Figure A.2), are given by the relation:

$$\begin{array}{l}\text{Summation of} \\ \text{Forces Acting on} \\ \text{Channel Element}\end{array} = F_p - \left(F_p + \frac{\partial F_p}{\partial x} \Delta x + F_1 + F_2\right) + F_3 + F_4 + F_5 \qquad (A.24)$$

where: F_p = hydrostatic force on upstream side of channel element;

$F_p + \frac{\partial F_p}{\partial x} \Delta x$ = hydrostatic force on downstream side of channel element;

F_1 and F_2 = bank forces resisting any unbalance in hydrostatic forces due to the widening or narrowing of the channel;

F_3 = friction force between water and boundary of channel element;

F_4 = gravity force in direction of flow; and,

F_5 = friction force acting on water due to wind.

The hydrostatic force on a section of the channel element is given by the relation:

$$F_p = \int_0^y \rho g(y-\eta)B(\eta)d\eta \qquad (A.25)$$

See Figure A.2 for definition sketch.

Equation (A.25) may be differentiated by applying Leibnitz's rule to obtain

$$\Delta x \frac{\partial F_p}{\partial x} = \int_0^y \Delta x \rho g \frac{\partial y}{\partial x} B(\eta)d\eta + \int_0^y \Delta x \rho g(y-\eta) \frac{\partial B(\eta)}{\partial x} d\eta \qquad (A.26)$$

since y and B are dependent x while η is not. Also, since

$$\int_0^y B(\eta)d\eta = A \tag{A.27}$$

and, recognizing that

$$\Delta x \int_0^y \rho g(y-\eta) \frac{\partial(B(\eta))}{\partial x} d\eta = -(F_1 + F_2) \tag{A.28}$$

it follows that:

$$F_p - (F_p + \frac{\partial F_p}{\partial x} \Delta x + F_1 + F_2)$$

$$= F_p - (F_p + \rho g A \frac{\partial y}{\partial x} \Delta x - (F_1 + F_2) + F_1 + F_2) \tag{A.29}$$

$$= - \rho g A \frac{\partial y}{\partial x} \Delta x \tag{A.30}$$

The friction force between the water and boundary of the channel element is given by the relation:

$$F_3 = P \cdot \tau_0 \Delta x \tag{A.31}$$

where P = wetted perimeter of the channel section;

and τ_0 = average friction shear over Δx.

Friction shear, τ_0, is defined by the relation:

$$\tau_0 = -\gamma R S_f \tag{A.32}$$

where: γ = ρg, the specific weight of water; where g is the acceleration due to gravity;

R = A/p, the hydraulic radius; and

S_f = friction slope or the slope of the total energy line.

Friction slope, S_f, may be estimated using the Chezy-Manning Equation in the form:

$$S_f = \frac{n^2V^2}{R^{4/3}} \qquad (A.33)$$

where n = Manning's roughness.

Substituting Equation (A.32) into Equation (A.31) yields:

$$F_3 = -\rho gA\Delta xS_f \qquad (A.34)$$

The gravity force acting on the water in the direction of flow is given by the relation:

$$F_4 = \gamma A\Delta x \sin\theta \qquad (A.35)$$

Notice that

$$\sin\theta \cong \tan\theta \cong S_o \qquad (A.36)$$

where S_o = the slope of the channel bottom,

Equation (A.33) may be written as

$$F_4 = \rho gA\Delta xS_o \qquad (A.37)$$

The friction force acting on the free water surface due to wind is examined by Amorocho and Devries (1980) and Devries and Amorocho (1980). They suggest that

$$F_5 = B\Delta x\tau_w \qquad (A.38)$$

where τ_w is the wind shear stress at the water surface. The wind shear stress may be calculated by the relation

$$\tau_w = \rho C_{w2} | u_2 \cos \phi_w | u_2 \cos \phi_w \qquad (A.39)$$

where: u_2 = wind velocity, (positive if in downstream direction);

ϕ_w = acute angle between wind direction and direction of flow of water; and,

$$C_w = 0.00262[1 + \exp(- \frac{u_2 - 10.5}{1.11})]^{-1} + 0.00137 \qquad (A.40)$$

If wind slope, S_w, is defined as

$$S_w = \frac{\tau_w}{\gamma D} \qquad (A.41)$$

where D = A/B, the hydraulic depth; Equation (A.38) may be written as

$$F_5 = \rho g A \Delta x S_w \qquad (A.42)$$

Substituting Equations (A.30), (A.34), (A.37) and (A.42) into Equation (A.24) gives

$$\begin{array}{l}\text{Summation of}\\ \text{Forces Acting on}\\ \text{Channel Element}\end{array} = - \rho g A \frac{\partial y}{\partial x} \Delta x - \rho g A \Delta x S_f + \rho g A \Delta x S_o$$

$$+ \rho g A \Delta x S_w \qquad (A.43)$$

Substituting Equations (A.23) and (A.43) into Equation (A.18) and recognizing that

$$\begin{array}{l}\text{Rate of}\\ \text{Accummulation of}\\ \text{Momentum in}\\ \text{Channel Element}\end{array} = \rho \frac{\partial(VA)}{\partial t} \Delta x \qquad (A.44)$$

the following form of the momentum equation results:

$$\frac{\partial(VA)}{\partial t} + \frac{\partial(V(VA))}{\partial x} + Ag \frac{\partial y}{\partial x} + Vq_o - V_i q_i$$

$$+ A_g S_f - A_g S_o - A_g S_w = 0 \qquad (A.45)$$

If the first two terms on the left side of Equation (A.45) are expanded,

$$V \frac{\partial A}{\partial t} + A \frac{\partial V}{\partial t} + V \frac{\partial(VA)}{\partial x} + VA \frac{\partial V}{\partial x} + Ag \frac{\partial y}{\partial x}$$

$$+ q_o V - q_i V_i + Ag(S_f - S_o - S_w) = 0 \qquad (A.46)$$

Multiplying Equation (A.8) by the average velocity, V, and substituting this result into Equation (A.46) yields:

$$V(q_i - q_o + p - i) + A \frac{\partial V}{\partial t} + VA \frac{\partial V}{\partial x} + Ag \frac{\partial y}{\partial x}$$

$$+ q_o V - q_i V_i + Ag(S_f - S_o - S_w) = 0 \qquad (A.47)$$

Dividing Equation (A.47) by cross-sectional area, A, and gathering terms yields:

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial y}{\partial x} + g(S_f - S_o - S_w)$$

$$+ \frac{q_i}{A} (V - V_i) + \frac{V}{A} (p-i) = 0 \qquad (A.48)$$

Several other useful forms of the momentum equation may be derived. Noting that Q = VA, Equation (A.45) may be written in terms of discharge and area as:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + Ag\,\frac{\partial y}{\partial x} + \frac{Q}{A}\,q_o - q_i V_i$$

$$+ Ag(S_f - S_o - S_w) = 0 \qquad (A.49)$$

which may be expanded to yield

$$2\,\frac{Q}{A}\,\frac{\partial Q}{\partial x} + \frac{\partial Q}{\partial t} - B\left(\frac{Q^2}{A^2}\right)\frac{\partial y}{\partial x} + Ag\,\frac{\partial y}{\partial x} - \left(\frac{Q^2}{A^2}\right) A_x^y$$

$$+ \frac{Q}{A}\,q_o - q_i V_i + Ag(S_f - S_o - S_w) = 0 \qquad (A.50)$$

where $A_x^y = \left.\frac{\partial A}{\partial x}\right| y = \text{constant}$. For prismatic channels $A_x^y = 0$.

Note that the Froude number, F, for open channel flow may be given by the relation:

$$F^2 = \frac{V^2}{(Ag/B)} = \frac{BV^2}{Ag} = \frac{B}{Ag}\left(\frac{Q^2}{A^2}\right) \qquad (A.51)$$

Dividing Equation (A.50) by g and A and substituting Equation (A.51), the following form of the momentum equation results:

$$2\,\frac{Q}{A^2 g}\,\frac{\partial Q}{\partial x} + \frac{1}{Ag}\,\frac{\partial Q}{\partial t} + (1 - F^2)\,\frac{\partial y}{\partial x} - \frac{1}{Ag}\left(\frac{Q^2}{A^2}\right) A_x^y$$

$$+ \frac{Q}{A^2 g}\,q_o - \frac{1}{Ag}\,q_i V_i + (S_f - S_o - S_w) = 0 \qquad (A.52)$$

Finally, if a wide rectangular prismatic channel is assumed; that is, $A = By$ and $A_x^y = 0$, Equation (A.45) may be written as:

$$\frac{\partial(Vy)}{\partial t} + \frac{\partial(V^2y)}{\partial x} + gy\,\frac{\partial y}{\partial x} + \frac{V}{B}\,q_o - \frac{V_i}{B}\,q_i$$

$$+\ gy(S_f - S_o - S_w) = 0 \qquad \text{(A.53)}$$

A.1.3 Summary of Basic Assumptions Implicit to the Derivation of the St. Venant Equation

The basic assumptions implicit to the derivation of the St. Venant Equations are as follows:

1. Vertical accelerations are negligible in comparison to horizontal accelerations and may be neglected.
2. Pressure distributions are hydrostatic.
3. Velocity distributions are uniform.
4. Friction resistance in unsteady flow is the same as the friction resistance in steady flow.
5. The channel slope is small. That is, the angle, θ, between the channel bed and the horizontal is sufficiently small that channel slope may be approximated by tan θ.

None of the these assumptions can be considered particularly restrictive when modelling gradually varying flow in irrigation canals.

A.2 Numerical Solution Using Implicit Finite Difference Techniques

Of the great variety of techniques used to solve the various forms of the St. Venant Equations, the one considered most suitable for application to modelling irrigation conveyance systems is the scheme introduced by Amein (1968) and demonstrated in Amein and Fang (1970). A thorough analysis of the numerical techniques introduced by Amein (1968) may be found in Fread (1973). A review and comparison of several implicit finite difference solutions to the St. Venant Equations including the one introduced by Amein may be found in Joliffe (1982).

The advantages of using the Amein four point implicit finite difference scheme to solve the St. Venant Equations include:

1. the ability to use relatively large time steps between calculations without accompanying loss of accuracy or development of instability, which has the effect of reducing computation costs;
2. the ability to use an irregular fixed grid system, which permits the investigator to easily select cross-sections of concern;
3. the fact that implicit finite difference solutions are among the most conservative of numerical techniques available, an important consideration where precise water balance calculations are required;
4. the solution method is established and available; and,
5. the solution method has been demonstrated to reliably reproduce field observations.

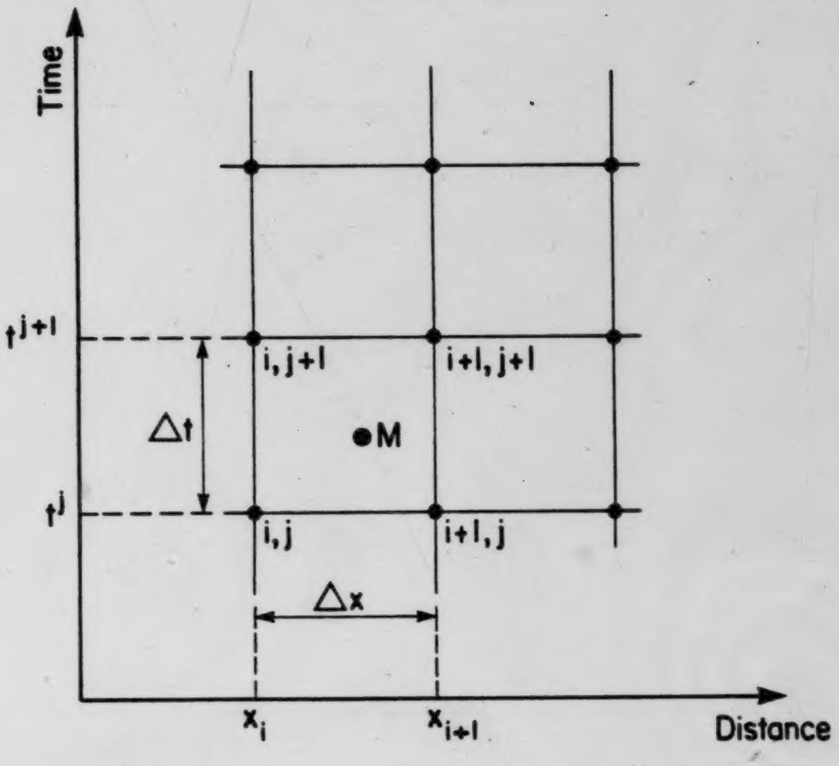

Time
t^{j+1}
t^j
Δt
i, j+1
i+1, j+1
M
i, j
i+1, j
Δx
x_i
x_{i+1}
Distance

Figure A.3 Distance-time grid for finite difference solution.

The particular form of the St. Venant Equations chosen for solution are given by Equations (A.15) and (A.48), and are restated here for convenience.

$$\frac{A}{B}\frac{\partial V}{\partial x} + V\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} - \frac{1}{B}(q_i - q_o + p - i) = 0 \qquad \text{(A.15)}$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} + g(S_f - S_o - S_w)$$

$$+ \frac{q_i}{A}(V - V_i) + \frac{V}{A}(p - i) = 0 \qquad \text{(A.48)}$$

Note that these equations are only applicable to prismatic channels. If it is further assumed that both bulk lateral inflow and bulk lateral outflow equal zero and that wind effects may be neglected, Equations (A.15) and (A.48) become:

$$\frac{A}{B}\frac{\partial V}{\partial x} + V\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} - \frac{q}{B} = 0 \qquad \text{(A.54)}$$

and;

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} + g(S_f - S_o) + \frac{V}{A}q = 0 \qquad \text{(A.55)}$$

where: $q = p - i$ (A.56)

The distance-time grid which will be used is shown in Figure A.3. Define any variable, α, at point m.

$$\alpha(m) = \frac{\theta}{2}\left(\alpha_{i+1}^{j+1} + \alpha_i^{j+1}\right) + \frac{(1 - \theta)}{2}\left(\alpha_{i+1}^{j} + \alpha_i^{j}\right) \qquad \text{(A.57)}$$

$$\frac{\partial \alpha(m)}{\partial x} = \theta\left(\frac{\alpha_{i+1}^{j+1} - \alpha_i^{j+1}}{\Delta x}\right) + (1 - \theta)\left(\frac{\alpha_{i+1}^{j} - \alpha_i^{j}}{\Delta x}\right) \qquad \text{(A.58)}$$

$$\frac{\partial \alpha(m)}{\partial t} = \frac{1}{2}\left(\frac{\alpha_{i+1}^{j+1} - \alpha_{i+1}^{j}}{\Delta t}\right) + \frac{1}{2}\left(\frac{\alpha_{i}^{j+1} - \alpha_{i}^{j}}{\Delta t}\right) \tag{A.59}$$

where θ is a weighting function which varies as $0.5 \leq \theta \leq 1.0$.

Substituting appropriate expressions for $\frac{\partial y}{\partial t}$, $\frac{A}{B}$, $\frac{\partial V}{\partial x}$, $\frac{\partial y}{\partial x}$, $\frac{q}{B}$, V, $\frac{\partial V}{\partial t}$ and S_f into Equations (A.54) and (A.55), rearranging and gathering terms yields:

$$\begin{aligned}
&(y_{i+1}^{j+1} + y_{i}^{j+1}) + C_1 + \frac{\Delta t}{\Delta x}\,\theta^2 \left[\left(\frac{A}{B}\right)_{i+1}^{j+1} + \left(\frac{A}{B}\right)_{i}^{j+1}\right]\left[V_{i+1}^{j+1} - V_{i}^{j+1}\right] \\
&+ \frac{\Delta t}{\Delta x}\,\theta\, C_2\left[V_{i+1}^{j+1} - V_{i}^{j+1}\right] + \frac{\Delta t}{\Delta x}\,\theta\, C_3\left[\left(\frac{A}{B}\right)_{i+1}^{j+1} + \left(\frac{A}{B}\right)_{i}^{j+1}\right] + C_4 \\
&+ \frac{\Delta t}{\Delta x}\,\theta^2\left[V_{i+1}^{j+1} + V_{i}^{j+1}\right]\left[y_{i+1}^{j+1} - y_{i}^{j+1}\right] + \frac{\Delta t}{\Delta x}\,\theta\, C_5\left[y_{i+1}^{j+1} - y_{i}^{j+1}\right] \\
&+ \frac{\Delta t}{\Delta x}\,\theta\, C_6\left[V_{i+1}^{j+1} + V_{i}^{j+1}\right] + C_7 - \Delta t\, q\, \theta\left[\left(\frac{1}{B}\right)_{i+1}^{j+1} + \left(\frac{1}{B}\right)_{i}^{j+1}\right] \\
&- C_8 = 0
\end{aligned} \tag{A.60}$$

where: $C_1 = -(y_{i+1}^{j} + y_{i}^{j})$ (A.61)

$$C_2 = (1 - \theta)\left[\left(\frac{A}{B}\right)_{i+1}^{j} + \left(\frac{A}{B}\right)_{i}^{j}\right] \tag{A.62}$$

$$C_3 = (1 - \theta)\left[V_{i+1}^{j} - V_{i}^{j}\right] \tag{A.63}$$

$$C_4 = \frac{\Delta t}{\Delta x}\, C_2\, C_3 \tag{A.64}$$

$$C_5 = (1 - \theta)\left[V_{i+1}^{j} + V_{i}^{j}\right] \tag{A.65}$$

$$C_6 = (1 - \theta)\left(y^j_{i+1} - y^j_i\right) \qquad \text{(A.66)}$$

$$C_7 = \frac{\Delta t}{\Delta x} C_5 C_6 \qquad \text{(A.67)}$$

and $$C_8 = \Delta t\ q\ (1 - \theta)\left((\tfrac{1}{B})^j_{i+1} + (\tfrac{1}{B})^j_i\right) \qquad \text{(A.68)}$$

and,

$$\frac{1}{g}\frac{\Delta x}{\Delta t}\left(V^{j+1}_{i+1} + V^{j+1}_i\right) + C_9 + \frac{\theta^2}{g}\left((V^{j+1}_{i+1})^2 - (V^{j+1}_i)^2\right.$$

$$+ C_{10}\frac{\theta}{g}\left(V^{j+1}_{i+1} - V^{j+1}_i\right) + C_{11}\frac{\theta}{g}\left(V^{j+1}_{i+1} + V^{j+1}_i\right) + C_{12}$$

$$+ 2\theta\left(y^{j+1}_{i+1} - y^{j+1}_i\right) + C_{13} + C_{14} + \Delta x\ \theta\left((S_f)^{j+1}_{i+1} + (S_f)^{j+1}_i\right)$$

$$+ C_{15} + q\frac{\Delta x}{g}\theta\left((\tfrac{V}{A})^{j+1}_{i+1} + (\tfrac{V}{A})^{j+1}_i\right) + C_{16} = 0 \qquad \text{(A.69)}$$

where

$$C_9 = -\frac{1}{g}\frac{\Delta x}{\Delta t}\left(V^j_{i+1} + V^j_i\right) \qquad \text{(A.70)}$$

$$C_{10} = (1 - \theta)\left(V^j_{i+1} + V^j_i\right) \qquad \text{(A.71)}$$

$$C_{11} = (1 - \theta)\left(V^j_{i+1} - V^j_i\right) \qquad \text{(A.72)}$$

$$C_{12} = \frac{(1 - \theta)^2}{g}\left((V^j_{i+1})^2 - (V^j_i)^2\right) \qquad \text{(A.73)}$$

$$C_{13} = 2(1 - \theta)\left(y^j_{i+1} - y^j_i\right) \qquad \text{(A.74)}$$

$$C_{14} = 2\left[z^{j}_{i+1} - z^{j}_{i}\right] \tag{A.75}$$

$$C_{15} = \Delta x(1 - \theta)\left[(S_f)^{j}_{i+1} + (S_f)^{j}_{i}\right] \tag{A.76}$$

$$C_{16} = q\,\frac{\Delta x}{g}\,(1 - \theta)\left[\left(\frac{V}{A}\right)^{j}_{i+1} + \left(\frac{V}{A}\right)^{j}_{i}\right] \tag{A.77}$$

and $$(S_f)^{j}_{i} = \frac{(n^{j}_{i})^{2} V^{j}_{i}\ |V^{j}_{i}|\ (P^{j}_{i})^{4/3}}{(A^{j}_{i})^{4/3}} \tag{A.78}$$

where P^{j}_{i} = the wetted perimeter. $(S_f)^{j+1}_{i}$, $(S_f)^{j}_{i+1}$, and $(S_f)^{j+1}_{i+1}$ are calculated in a similar manner.

Equations (A.60) and (A.69) may be written as:

$$\begin{aligned} F_1\left[y^{j+1}_{i}, V^{j+1}_{i}, y^{j+1}_{i+1}, V^{j+1}_{i+1}\right] &= \left[y^{j+1}_{i+1} + y^{j+1}_{i}\right] + C_1 \\ &+ \frac{\Delta t}{\Delta x}\,\theta^{2}\left[\left(\frac{A}{B}\right)^{j+1}_{i+1} + \left(\frac{A}{B}\right)^{j+1}_{i}\right]\left[V^{j+1}_{i+1} - V^{j+1}_{i}\right] \\ &+ \frac{\Delta t}{\Delta x}\,\theta C_2\left[V^{j+1}_{i+1} - V^{j+1}_{i}\right] + \frac{\Delta t}{\Delta x}\,\theta\, C_3\left[\left(\frac{A}{B}\right)^{j+1}_{i+1} + \left(\frac{A}{B}\right)^{j+1}_{i}\right] + C_4 \\ &+ \frac{\Delta t}{\Delta x}\,\theta^{2}\left[V^{j+1}_{i+1} + V^{j+1}_{i}\right]\left[y^{j+1}_{i+1} - y^{j+1}_{i}\right] + \frac{\Delta t}{\Delta x}\,\theta\, C_5\left[y^{j+1}_{i+1} - y^{j+1}_{i}\right] \\ &+ \frac{\Delta t}{\Delta x}\,\theta\, C_6\left[V^{j+1}_{i+1} + V^{j+1}_{i}\right] + C_7 \\ &- \Delta t\, q\, \theta\left[\left(\frac{1}{B}\right)^{j+1}_{i+1} + \left(\frac{1}{B}\right)^{j+1}_{i}\right] - C_8 = 0 \end{aligned} \tag{A.79}$$

and

$$G_i\left(y_i^{j+1}, V_i^{j+1}, y_{i+1}^{j+1}, V_{i+1}^{j+1}\right) = \frac{1}{g}\frac{\Delta x}{\Delta t}\left(V_{i+1}^{j+1} + V_i^{j+1}\right)$$

$$+ C_9 + \frac{\theta^2}{g}\left[(V_{i+1}^{j+1})^2 - (V_i^{j+1})^2\right] + C_{10}\,\frac{\theta}{g}\left(V_{i+1}^{j+1} - V_i^{j+1}\right)$$

$$+ C_{11}\,\frac{\theta}{g}\left(V_{i+1}^{j+1} + V_i^{j+1}\right) + C_{12} + 2\theta\left(y_{i+1}^{j+1} - y_i^{j+1}\right) + C_{13} + C_{14}$$

$$+ \Delta x\ \theta\left[(S_f)_{i+1}^{j+1} + (S_f)_i^{j+1}\right] + C_{15}$$

$$+ q\,\frac{\Delta x}{g}\,\theta\left[\left(\frac{V}{A}\right)_{i+1}^{j+1} + \left(\frac{V}{A}\right)_i^{j+1}\right] + C_{16} = 0 \qquad (A.80)$$

respectively.

Solution of Equations (A.79) and (A.80) permit the simulation of gradually varied unsteady flow conditions in irregular shaped prismatic channels. If there are N interior points, 2(N - 1) equations may be written to solve for 2N unknowns. The remaining two equations are supplied by the boundary conditions at the inlet and outlet of the channel. The 2N equations may then be written as:

$$G_o\left(y_1^{j+1},\ V_1^{j+1}\right) = 0$$

$$F_1\left(y_1^{j+1},\ V_1^{j+1},\ y_2^{j+1},\ V_2^{j+1}\right) = 0$$

$$G_1\left(y_1^{j+1},\ V_1^{j+1},\ y_2^{j+1},\ V_2^{j+1}\right) = 0$$

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$$F_i\left(y_i^{j+1},\ V_i^{j+1},\ y_{i+1}^{j+1},\ V_{i+1}^{j+1}\right) = 0$$

$$G_i\left(y_i^{j+1},\ V_i^{j+1},\ y_{i+1}^{j+1},\ V_{i+1}^{j+1}\right) = 0$$

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$$F_{N-1}\left(y_{N-1}^{j+1},\ V_{N-1}^{j+1},\ y_N^{j+1},\ V_N^{j+1}\right) = 0$$

$$G_{N-1}\left(y_{N-1}^{j+1},\ V_{N-1}^{j+1},\ y_N^{j+1},\ V_N^{j+1}\right) = 0$$

$$F_N\left(y_N^{j+1},\ V_N^{j+1}\right) = 0 \tag{A.81}$$

At time t^j all of the flow variables are known. Determination of all of the flow variables at time $t^{j+1} = t^j + \Delta t$ simply requires the solution of Equation (A.81).

Equation (A.81) represents a collection of non-linear equations which may be solved by application of the generalized Newton iteration method. Estimates of the unknown variables are substituted into the equation. The value of the equation may not be zero, but acquire values known as residuals. Equation (A.81) is solved by adjusting the variables until the residuals either equal zero or are an acceptably low value. The Newton iteration method provides a rapidly converging method for achieving the solution. The method is as follows:

Let the residuals of the kth iteration by represented by $r_{1,i}^{k}$ and $r_{2,i}^{k}$ for the F and G functions respectively. The values of the residuals for the kth iteration cycle are given by the following equation:

$$G_o\left(y_{1,k}^{j+1}, V_{1,k}^{j+1}\right) = r_{2,0}^{k}$$

$$F_1\left(y_{1,k}^{j+1}, V_{1,k}^{j+1}, y_{2,k}^{j+1}, V_{2,k}^{j+1}\right) = r_{1,1}^{k}$$

$$G_1\left(y_{1,k}^{j+1}, V_{1,k}^{j+1}, y_{2,k}^{j+1}, V_{2,k}^{j+1}\right) = r_{2,1}^{k}$$

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$$F_i\left(y_{i,k}^{j+1}, V_{i,k}^{j+1}, y_{i+1,k}^{j+1}, V_{i+1,k}^{j+1}\right) = r_{1,i}^{k}$$

$$G_i\left(y_{i,k}^{j+1}, V_{i,k}^{j+1}, y_{i+1,k}^{j+1}, V_{i+1,k}^{j+1}\right) = r_{2,i}^{k}$$

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$$F_{N-1}\left(y_{N-1,k}^{j+1}, V_{N-1,k}^{j+1}, y_{N,k}^{j+1}, V_{N,k}^{j+1}\right) = r_{1,N-1}^{k}$$

$$G_{N-1}\left(y_{N-1,k}^{j+1}, V_{N-1,k}^{j+1}, y_{N,k}^{j+1}, V_{N,k}^{j+1}\right) = r_{2,N-1}^{k}$$

$$F_N\left(y_{N,k}^{j+1}, V_{N,k}^{j+1}\right) = r_{1,N}^{k} \qquad \text{(A.82)}$$

The residuals and the partial derivatives of Equation (A.81) are related by the following equation:

$$\frac{\partial G_o}{\partial y_1^{j+1}} dy_1^{j+1} + \frac{\partial G_o}{\partial V_1^{j+1}} dV_1^{j+1} = -r_{2,0}^k$$

$$\frac{\partial F_1}{\partial y_1^{j+1}} dy_1^{j+1} + \frac{\partial F_1}{\partial V_1^{j+1}} dV_1^{j+1} + \frac{\partial F_1}{\partial y_2^{j+1}} dy_2^{j+1} + \frac{\partial F_1}{\partial V_2^{j+1}} dV_2^{j+1} = -r_{1,1}^k$$

$$\frac{\partial G_1}{\partial y_1^{j+1}} dy_1^{j+1} + \frac{\partial G_1}{\partial V_1^{j+1}} dV_1^{j+1} + \frac{\partial G_1}{\partial y_2^{j+1}} dy_2^{j+1} + \frac{\partial G_1}{\partial y_2^{j+1}} dV_2^{j+1} = -r_{2,1}^k$$

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$$\frac{\partial F_i}{\partial y_i^{j+1}} dy_i^{j+1} + \frac{\partial F_i}{\partial V_i^{j+1}} dV_i^{j+1} + \frac{\partial F_i}{\partial y_{i+1}^{j+1}} dy_{i+1}^{j+1} + \frac{\partial F_i}{\partial V_{i+1}^{j+1}} dV_{i+1}^{j+1} = -r_{1,i}^k$$

$$\frac{\partial G_i}{\partial y_i^{j+1}} dy_i^{j+1} + \frac{\partial G_i}{\partial V_i^{j+1}} dV_i^{j+1} + \frac{\partial G_i}{\partial y_{i+1}^{j+1}} dy_{i+1}^{j+1} + \frac{\partial G_i}{\partial V_{i+1}^{j+1}} dV_{j+1}^{j+1} = -r_{2,i}^k$$

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$$\frac{\partial F_{N-1}}{\partial y_{N-1}^{j+1}} dy_{N-1}^{j+1} + \frac{\partial F_{N-1}}{\partial V_{N-1}^{j+1}} dV_{N-1}^{j+1} + \frac{\partial F_{N-1}}{\partial y_N^{j+1}} dy_N^{j+1} + \frac{\partial F_{N-1}}{\partial V_N^{j+1}} dV_N^{j+1} = -r_{1,N-1}^k$$

$$\frac{\partial G_{N-1}}{\partial y_{N-1}^{j+1}} dy_{N-1}^{j+1} + \frac{\partial G_{N-1}}{\partial V_{N-1}^{j+1}} dV_{N-1}^{j+1} + \frac{\partial G_{N-1}}{\partial y_N^{j+1}} dy_N^{j+1} + \frac{\partial G_{N-1}}{\partial V_N^{j+1}} dV_N^{j+1} = -r_{2,N-1}^k$$

$$\frac{\partial F_N}{\partial y_N^{j+1}} dy_N^{j+1} + \frac{\partial F_N}{\partial V_N^{j+1}} dV_N^{j+1} = -r_{1,N}^k \qquad (A.83)$$

where:

$$dy_1^{j+1} = y_{1,k+1}^{j+1} - y_{1,k}^{j+1}$$

$$dV_1^{j+1} = V_{1,k+1}^{j+1} - V_{1,k}^{j+1}$$

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$$dy_i^{j+1} = y_{i,k+1}^{j+1} - y_{i,k}^{j+1}$$

$$dV_i^{j+1} = V_{i,k+1}^{j+1} - V_{i,k}^{j+1}$$

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$$dy_N^{j+1} = y_{N,k+1}^{j+1} - y_{N,k}^{j+1}$$

$$dV_N^{j+1} = V_{N,k+1}^{j+1} - V_{N,k}^{j+1} \qquad \text{(A.84)}$$

and the partial derivatives are evaluated using the relations:

$$\frac{\partial F_i}{\partial y_i^{j+1}} = 1 + \frac{\Delta t}{\Delta x}\,\theta^2\left(1 - \frac{A_i^{j+1}}{(B_i^{j+1})^2}\,\frac{dB_i^{j+1}}{dy_i^{j+1}}\right)\left(V_{i+1}^{j+1} - V_i^{j+1}\right)$$

$$+ \frac{\Delta t}{\Delta x}\,\theta\, C_3\left(1 - \frac{A_i^{j+1}}{(B_i^{j+1})^2}\,\frac{dB_i^{\,j+1}}{dy_i^{j+1}}\right) - \frac{\Delta t}{\Delta x}\,\theta^2\left(V_{i+1}^{j+1} + V_i^{j+1}\right)$$

$$- \frac{\Delta t}{\Delta x}\,\theta\, C_5 + \Delta t\, q\, \theta\left(\frac{1}{(B_i^{j+1})^2}\,\frac{dB_i^{j+1}}{dy_i^{j+1}}\right) \qquad \text{(A.85)}$$

$$\frac{\partial F_i}{\partial y_{i+1}^{j+1}} = 1 + \frac{\Delta t}{\Delta x}\,\theta^2\left(1 - \frac{A_{i+1}^{j+1}}{(B_{i+1}^{j+1})^2}\,\frac{dB_{i+1}^{j+1}}{dy_{i+1}^{j+1}}\right)\left(V_{i+1}^{j+1} - V_i^{j+1}\right)$$

$$+ \frac{\Delta t}{\Delta x}\,\theta\,C_3\left(1 - \frac{A_{i+1}^{j+1}}{(B_{i+1}^{j+1})^2}\,\frac{dB_{i+1}^{j+1}}{dy_{i+1}^{j+1}}\right) + \frac{\Delta t}{\Delta x}\,\theta^2\left(V_{i+1}^{j+1} + V_i^{j+1}\right)$$

$$+ \frac{\Delta t}{\Delta x}\,\theta\,C_5 + \Delta t\,q\,\theta\left(\frac{1}{(B_{i+1}^{j+1})^2}\,\frac{dB_{i+1}^{j+1}}{dy_{i+1}^{j+1}}\right) \qquad (A.86)$$

$$\frac{\partial F_i}{\partial V_i^{j+1}} = -\frac{\Delta t}{\Delta x}\,\theta^2\left(\left(\frac{A}{B}\right)_{i+1}^{j+1} + \left(\frac{A}{B}\right)_i^{j+1}\right) - \frac{\Delta t}{\Delta x}\,\theta\,C_2$$

$$+ \frac{\Delta t}{\Delta x}\,\theta^2\left(y_{i+1}^{j+1} - y_i^{j+1}\right) + \frac{\Delta t}{\Delta x}\,\theta\,C_6 \qquad (A.87)$$

$$\frac{\partial F_i}{\partial V_{i+1}^{j+1}} = \frac{\Delta t}{\Delta x}\,\theta^2\left(\left(\frac{A}{B}\right)_{i+1}^{j+1} + \left(\frac{A}{B}\right)_i^{j+1}\right) + \frac{\Delta t}{\Delta x}\,\theta\,C_2$$

$$+ \frac{\Delta t}{\Delta x}\,\theta^2\left(y_{i+1}^{j+1} - y_i^{j+1}\right) + \frac{\Delta t}{\Delta x}\,\theta\,C_6 \qquad (A.88)$$

$$\frac{\partial G_i}{\partial y_i^{j+1}} = -2\theta + \frac{4}{3}\,\Delta x\,\theta\,(S_f)_i^{j+1}\left(\frac{1}{P_i^{j+1}}\,\frac{dP_i^{j+1}}{dy_i^{j+1}} - \frac{B_i^{j+1}}{A_i^{j+1}}\right)$$

$$-\,q\,\frac{\Delta x}{g}\,\theta\,\frac{V_i^{j+1}B_i^{j+1}}{(A_i^{j+1})^2} \qquad (A.89)$$

$$\frac{\partial G_i}{\partial y_{i+1}^{j+1}} = 2\theta + \frac{4}{3}\,\Delta x\;\theta\;(S_f)_{i+1}^{j+1}\left(\frac{1}{P_{i+1}^{j+1}}\,\frac{dP_{i+1}^{j+1}}{dy_{i+1}^{j+1}} - \frac{B_{i+1}^{j+1}}{A_{i+1}^{j+1}}\right)$$

$$- q\,\frac{\Delta x}{g}\;\theta\;\frac{V_{i+1}^{j+1}\;B_{i+1}^{j+1}}{(A_{i+1}^{j+1})^2} \tag{A.90}$$

$$\frac{\partial G_i}{\partial V_i^{j+1}} = \frac{1}{g}\,\frac{\Delta x}{\Delta t} - \frac{2\theta^2}{g}\,V_i^{j+1} - C_{10}\,\frac{\theta}{g} + C_{11}\,\frac{\theta}{g}$$

$$+ 2\Delta x\;\theta\;\frac{(S_f)_i^{j+1}}{V_i^{j+1}} + q\,\frac{\Delta x}{g}\,\frac{\theta}{A_i^{j+1}} \tag{A.91}$$

and

$$\frac{\partial G_i}{\partial V_{i+1}^{j+1}} = \frac{1}{g}\,\frac{\Delta x}{\Delta t} + \frac{2\theta^2}{g}\,V_{i+1}^{j+1} + C_{10}\,\frac{\theta}{g} + C_{11}\,\frac{\theta}{g}$$

$$+ 2\Delta x\;\theta\;\frac{(S_f)_{i+1}^{j+1}}{V_{i+1}^{j+1}} + q\,\frac{\Delta x}{g}\,\frac{\theta}{A_{i+1}^{j+1}} \tag{A.92}$$

Note also that if B_i and P_i are known continuous functions of y_i, it is possible to determine:

$$\frac{dB_i^{j+1}}{dy_i^{j+1}} = \frac{dB_i}{dy_i} \quad \text{evaluated at } y_i^{j+1} \tag{A.93}$$

$$\frac{dB_{i+1}^{j+1}}{dy_{i+1}^{j+1}} = \frac{dB_i}{dy_i} \quad \text{evaluated at } y_{i+1}^{j+1} \tag{A.94}$$

$$\frac{dP_i^{j+1}}{dy_i^{j+1}} = \frac{dP_i}{dy_i} \quad \text{evaluated at } y_i^{j+1} \tag{A.95}$$

and

$$\frac{dP_{i+1}^{j+1}}{dy_{i+1}^{j+1}} = \frac{dP_i}{dy_i} \quad \text{evaluated at } y_{i+1}^{j+1} \tag{A.96}$$

Estimates of the unknown flow variables at t^{j+1} are made and the residuals are calculated using Equation (A.82). These are then substituted into Equation (A.83) which is then solved for the estimates of the unknown flow variables. These are then substituted back into Equation (A.82) and a new set of residuals is calculated. If these residuals equal zero or are approximately so, the flow variables are accepted as correct and the time step can change. If the residuals are still too large, the iteration is repeated.

It should be noted that Equation (A.83) is not that difficult to solve. It is a sparse diagonal matrix for which several methods of solution are available.

The numerical solution outlined requires that at least two additional equations describing the inlet and outlet boundary conditions be known. Inlet boundary conditions must be either stage or discharge as function of time; that is,

$$Q_1 = f(t) \tag{A.97}$$

or $$h_1 = f(t) \tag{A.98}$$

where Q_1 is the discharge at the canal inlet and h_1 is the stage at the canal inlet. Outlet boundary conditions may be either discharge as a function of canal stage (a rating curve) discharge as a function of time, or stage as a function of time. In equation form

$$Q_0 = f(h_0) \tag{A.99}$$

$$Q_0 = f(h_0,t) \tag{A.100}$$

$$Q_0 = f(t) \tag{A.101}$$

$$h_0 = f(t) \tag{A.102}$$

where Q_0 is the discharge of the canal at the outlet and h_0 is the stage of the canal immediately upstream of the diversion structure. Note that Equation (A.100) describes an outlet boundary condition which is a function of time. This type of boundary condition is very common in irrigation conveyance systems.

The boundary conditions of canals within irrigation conveyance systems are relatively simple to identify and present no difficulties in their formulation. They are determined by the physical nature and method of operation of the hydraulic structures.

If a canal has numerous structures within it (as is usually the case) it is divided into reaches bounded on either end by a suitable structure. Calculations would proceed downstream starting with the upstream most reach.

Many hydraulic structures may allow downstream flow characteristics to influence upstream flow conditions. These structures do not provide adequate boundary conditions for numerical solution as previously described. The canal reaches separated by structures of this type must be considered simultaneously with the structure. A suitable outlet boundary condition is still required. Flow equations of hydraulic structures which allow downstream flow characteristics to influence upstream flow conditions include at least the following types:

$$Q_{u1} = Q_{d1} = f(\Delta h_{u1,d1}) \tag{A.103}$$

$$Q_{u1} = Q_{d1} = f(\Delta h_{u1,d1},t) \tag{A.104}$$

$$Q_{d1} = Q_{u1} + f(t) \tag{A.105}$$

$$Q_{d1} = Q_{u1} - f(t) \tag{A.106}$$

$$Q_{d1} = Q_{u1} + f(\Delta h_{u2,d1}) \tag{A.107}$$

$$Q_{d1} = Q_{u1} - f(\Delta h_{u1,d2}) \tag{A.108}$$

$$Q_{d1} = Q_{u1} + f(\Delta h_{u2,d1},t) \tag{A.109}$$

$$Q_{d1} = Q_{u1} - f(\Delta h_{u1,d2},t) \tag{A.110}$$

$$Q_{d1} = Q_{u1} = f(h_{d1}) \tag{A.111}$$

where:

Q_{u1} = the discharge upstream of the control structure,

Q_{d1} = the discharge downstream of the control structure,

$\Delta h_{u1,d1}$ = the upstream and downstream stage of the water in the same canal

$\Delta h_{u2,u1}$ = the difference between the inlet canal stage and the receiving canal stage,

$\Delta h_{u1,d2}$ = the difference between the outlet canal stage to the receiving canal stage, and

h_{d1} = the downstream stage in canal being studied.

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APPENDIX B

NEWTON ITERATIVE SOLUTION FOR THE ONE-DIMENSIONAL STEADY SPATIALLY VARIED FLOW EQUATIONS EXPRESSED IN FINITE DIFFERENCE FORM

APPENDIX B

NEWTON ITERATIVE SOLUTION FOR THE ONE-DIMENSIONAL STEADY SPATIALLY VARIED FLOW EQUATIONS EXPRESSED IN FINITE DIFFERENCE FORM

B.1 Introduction

A technique for calculating open-channel flow profiles by solving the gradually varied flow equations of steady spatially constant flow, using Newton's Iteration Method, was introduced by Fread and Harbaugh (1971). Later, Fread (1973) employed this technique to generate initial flow conditions in a study he performed on the implicit finite difference solution of the St. Venant Equations discussed in Appendix A. The same numerical solution technique used by Fread and Harbaugh (1971) is used to calculate flow profiles of steady, spatially varied open channel flow which is described by equations derived and discussed by Henderson (1966).

B.2 Gradually Varied Flow Equations for Steady Spatially Varied Flow Conditions

B.2.1 Lateral Inflow

The equation describing steady gradually varied flow with lateral inflow may be developed using momentum considerations, Henderson (1966). The rate of change of momentum at a particular cross-section is described by the equation:

$$\frac{dM}{dx} = A(S_o - S_f) \tag{B.1}$$

where: M = momentum;

x = distance along channel;

A = cross-sectional area;

S_o = slope of channel bottom; and,

S_f = slope of total energy line.

Recognizing that

$$M = \frac{Q^2}{gA} + A\overline{y} \tag{B.2}$$

and that $d(A\overline{y})/dy = A$ (B.3)

it can be shown that the steady gradually varied flow equation may be written as:

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{2Q}{gA^2} q_i}{1 - F^2} \tag{B.4}$$

where: Q = rate of flow in channel;

q_i = rate of lateral inflow;

$\overline{y}$ = distance of water surface to centroid of channel cross-sectional area;

y = channel depth;

F = Froude Number; and,

g = acceleration due to gravity.

Noting that

$$F^2 = \frac{Q^2 B}{gA^3} \tag{B.5}$$

where B = top width of water surface; and that $\frac{dA}{dx} = Bdy$, (B.6)

Equation (B.4) may be written as

$$\frac{dy}{dx} + \frac{2Q}{gA^2} q_i - \frac{Q^2}{gA^3}\frac{dA}{dx} = S_o - S_f \tag{B.7}$$

B.2.2 Lateral Outflow

The equation describing steady gradually varied flow with lateral outflow may be developed using energy considerations, Henderson (1966). The rate of change of energy at a particular cross-section is described by the equation:

$$\frac{dE}{dx} = S_o - S_f \tag{B.8}$$

where: E = specific energy;

x = distance along channel;

S_o = slope of channel bottom; and,

S_f = slope of total energy line.

Recognizing that

$$E = \frac{Q^2}{2gA^2} + y \tag{B.9}$$

it can readily be shown that

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{Q}{gA^2} q_o}{1 - F^2} \tag{B.10}$$

where: Q = rate of flow in channel;

q_o = rate of lateral outflow;

y = channel depth;

F = Froude Number; and,

g = acceleration due to gravity.

Equation (B.10) may be rewritten in similar manner to that used to develop Equation (B.7) to yield

$$\frac{dy}{dx} + \frac{Q}{gA^2} q_o - \frac{Q^2}{gA^3} \frac{dA}{dx} = S_o - S_f \tag{B.11}$$

B.3 Newton's Iteration Method

(The following description is excerpted, with minor deletions, from Fread and Harbaugh (1971)).

Suppose an expression $y = f(x)$ is known and it is desired to find the value of x which will make $y = 0$. This value of x is denoted as X. An iterative technique, Newton's Iterative Method, may be used to find a solution to the equation, $f(x) = 0$. In this method, $f(x)$ is expanded in a Taylor Series about the point, x_1, which is an initial approximation of X. Thus,

$$y = f(x) = f(x_1) + \frac{f'(x_1)(x - x_1)}{1!} + \frac{f''(x_1)(x - x_1)^2}{2!} + \ldots \qquad (B.12)$$

The prime, ('), denotes a derivative; i.e., $f'(x_1) = df(x_1)/dx_1$. Since $f(x) = 0$, and if second order terms of the Taylor Series expansion in Equation (B.12) are neglected, then

$$f(x_1) + f'(x_1)(x - x_1) = 0 \qquad (B.13)$$

Solving equation (13) for x yields

$$x = x_1 - \frac{f(x_1)}{f'(x_1)} \qquad (B.14)$$

This value of x is an approximation of X, which is the value being sought, since the second and higher terms of the Taylor Series were discarded. However, if x is now substituted for x_1 in Equation (B.14), the resulting value of x will converge toward X. Thus, successive iterations of Equation (B.14) will produce a value of x which is sufficiently close to X. A general iterative formula for this procedure is given by

$$x^{k+1} = x^k - \frac{f(x^k)}{f'(x^k)} \tag{B.15}$$

where the superscript k denotes the number of iterations. After choosing an appropriate starting value, x^1, and applying Equation (B.15), the value of x will be found such that $f(x) = 0$. The initial approximation x^1 must be chosen sufficiently close to x. It can be shown, Conte and de Boor (1980), that if x^1 satisfies the inequality

$$\frac{|f(x^1)f''(x^1)|}{[f'(x^1)]^2} < 1 \tag{B.16}$$

where the double prime superscript denotes the second derivative, then x^k will converge quadratically to the root X of the equation $f(x) = 0$ upon applying Equation (B.15).

Convergence is attained when either

$$|x^{k+1} - x^k| < E_1 \tag{B.17}$$

or $$|f(x^k)| < E_2 \tag{B.18}$$

where E_1 and E_2 are error tolerances, that is, the desired absolute accuracy of the solution X of the equation $f(x) = 0$.

B.4 Application of Newton's Iterative Method to Determine Flow Profiles

Consider the case where there is distributed lateral outflow. Equation (B.11) may be rearranged to yield

$$\frac{dy}{dx} + \frac{Q}{gA^2} q_o - \frac{Q^2}{gA^3} \frac{dA}{dx} - S_o + S_f = 0 \tag{B.19}$$

Equation (B.19) may be expressed in difference form to yield

$$\frac{y_{i+1} - y_i}{\Delta x} + \frac{1}{g} \cdot \frac{1}{2} \quad \frac{Q_{i+1}}{A_{i+1}^2} + \frac{Q_i}{A_i^2} \quad \cdot q_o$$

$$- \frac{1}{2g} \quad \frac{Q_{i+1}^2}{A_{i+1}^3} + \frac{Q_i^2}{A_i^3} \quad \cdot \quad \frac{A_{i+1} - A_i}{\Delta x}$$

$$+ \frac{(Z_{i+1} - Z_i)}{\Delta x} + \frac{1}{2} \frac{Q_{i+1}^2 \; n^2 \; P_{i+1}^{4/3}}{A_{i+1}^{10/3}}$$

$$+ \frac{1}{2} \frac{Q_i^2 \; n^2 \; P_i^{4/3}}{A_i^{10/3}} = 0 \tag{B.20}$$

Multiplying Equation (B.20) by Δx and rearranging terms, we obtain;

$$y_{i+1} - y_i + \frac{\Delta x}{2g} \cdot q_o \frac{Q_{i+1}}{A_{i+1}^2} + \frac{\Delta x}{2g} \cdot q_o \frac{Q_i}{A_i^2}$$

$$- \frac{1}{2g} \frac{Q_{i+1}^2}{A_{i+1}^2} - \frac{Q_i^2 A_{i+1}}{2gA_i^3} + \frac{Q_{i+1}^2 A_i}{2gA_{i+1}^3} + \frac{Q_i^2}{2gA_i^2}$$

$$+ Z_{i+1} - Z_i + \frac{\Delta x}{2} \frac{Q_{i+1}^2 \; n^2 \; P_{i+1}^{4/3}}{A_{i+1}^{10/3}} + \frac{\Delta x}{2} \frac{Q_i^2 \; n^2 \; P_i^{4/3}}{A_i^{10/3}} = 0 \qquad (B.21)$$

Equation (B.21) is now in the form f(x) = 0 which may be solved using Newton's Iterative Method. That is,

$$f(y_i) = -y_{i+1} + y_i - \frac{\Delta x}{2g} q_o \left(\frac{Q_{i+1}}{A_{i+1}^2} + \frac{Q_i}{A_i^2} \right)$$

$$+ \frac{Q_{i+1}^2}{2g} \left(\frac{1}{A_{i+1}^2} - \frac{A_i}{A_{i+1}^3} \right) + \frac{Q_i^3}{2g} \left(\frac{A_{i+1}}{A_i^3} - \frac{1}{A_i^2} \right)$$

$$- (Z_{i+1} - Z_i) - \frac{\Delta x n^2}{2} \left(\frac{Q_{i+1}^2 P_{i+1}^{4/3}}{A_{i+1}^{10/3}} + \frac{Q_i^2 P_i^{4/3}}{A_i^{10/3}} \right) \qquad (B.22)$$

which may be differentiated to yield

$$f'(y_i) = 1 + \frac{\Delta x}{g} q_o \, Q_i \frac{B_i}{A_i^3} - \frac{3}{2g} \frac{Q_i^2 B_i A_{i+1}}{A_i^4}$$

$$- \frac{1}{2g} \frac{Q_{i+1}^2 B_i}{A_{i+1}^3} + \frac{1}{g} \frac{Q_i^2 B_i}{A_i^3} - \frac{2}{3} \Delta x \, n^2 \frac{Q_i^2 P_i^{1/3}}{A_i^{10/3}} \frac{dP_i}{dy_i}$$

$$+ \frac{5}{3} \Delta x \, n^2 \frac{Q_i^2 B_i P_i^{4/3}}{A_i^{13/3}} \qquad (B.23)$$

where: q_o = distributed outflow;

$Q_i = Q_{i+1} - q\Delta x$ (B.24)

n = known Manning roughness coefficient for open channels;

Z_{i+1} and Z_i = known elevations of channel bottom;

Δx = known distance between locations i and i+1 along channel;

and,

A_i, A_{i+1}, B_i, B_{i+1}, P_i, P_{i+1} and $\frac{dP_i}{dy_i}$ may be calculated from

known functions of y.

The k+1 estimate of y_i is determined using the relation

$$y_i^{k+1} = y_i^k - \frac{f(y_i^k)}{f'(y_i^k)} \tag{B.25}$$

Note that the calculation will proceed upstream from some suitable downstream boundary condition. That is, the depth at location i+1 is always known and we are determining the depth at location i (see Fig. B.1). Once the depth at i is determined, as based on criteria indicated by Equations (B.17) and (B.18), the iteration stops and the depth at the next upstream location is determined. This process continues until the depths are known at all previously specified locations.

For the condition where the flux is distributed lateral inflow, q_0, is replaced by $2q_i$ and calculations proceed as before.

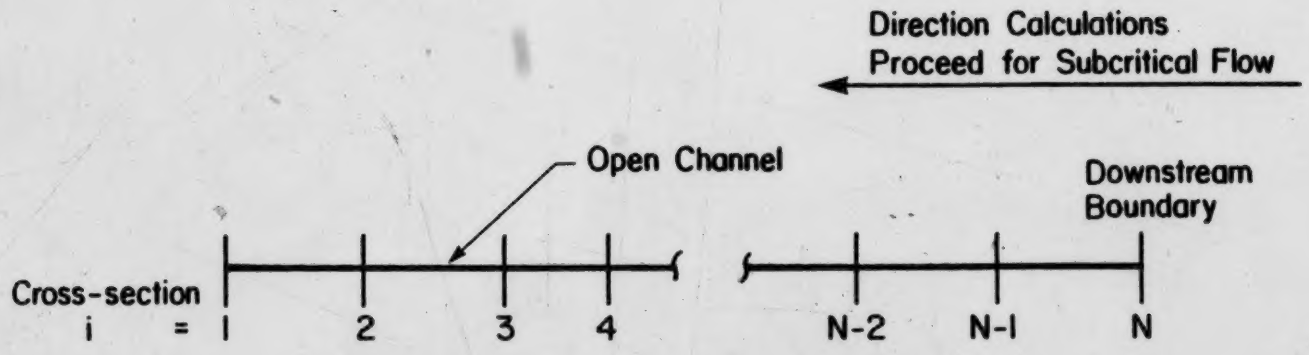
Direction Calculations
Proceed for Subcritical Flow
Open Channel
Downstream
Boundary
Cross-section
i = 1
2
3
4
N-2
N-1
N

Figure B.1 Procedure used to calculate backwater curves for subcritical flow.

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APPENDIX C

THE ALBERTA DEPARTMENT OF AGRICULTURE METHOD FOR IDENTIFICATION AND NAMING OF IRRIGATION CONVEYANCE SYSTEMS AND SUBSYSTEMS

APPENDIX C

THE ALBERTA DEPARTMENT OF AGRICULTURE METHOD FOR IDENTIFICATION AND NAMING OF IRRIGATION CONVEYANCE SYSTEMS AND SUBSYSTEMS

Most of the irrigated land area in Southern Alberta is organized into thirteen schemes called irrigation districts. (A description of the organization of irrigation districts typical of those found in Southern Alberta may be found in a report by J.A. Sagardoy titled Organization, Operation and Maintenance of Irrigation Schemes, FAO Paper No. 40, published by the Food and Agriculture Organization of the United Nations, 1982. In his report Sagardoy classifies irrigation districts as "irrigation associations"; that is, specialized water management organizations mainly controlled by farmers). The conveyance systems which serve the individual districts are owned, managed and operated by the farmers which receive the water. Nine conveyance systems supply water to the thirteen districts. The canals which first receive the water are called "mains" and are numbered one to nine.

Within any particular conveyance system submain canals may also be identified. These are canals which are considered to supply water to relatively distinct and significant portions of the conveyance system. Submain canals may receive water from the main canal or another submain. Submain canals are each assigned a number whose first digit is the number of the main canal supplying the conveyance system. Mains and submains may be further divided into blocks which are each assigned a letter of the alphabet.

An example of an irrigation conveyance system divided into main, submains and blocks is shown in Figure C.1. Main canal number 7 is divided into blocks A and B; submain 71 is divided into blocks A and B; submain 72 is divided into blocks A, B and C; and, submain 73 is divided into blocks A, B and C.

Portions of the conveyance system supplied with water from individual main or submain blocks are identified using the alpha-numeric system illustrated in Figure C.2. Only those portions of the conveyance system owned and operated by the irrigation district are considered. The locations where conveyance system operation and ownership changes from the district to the farmer are indicated by arrows.

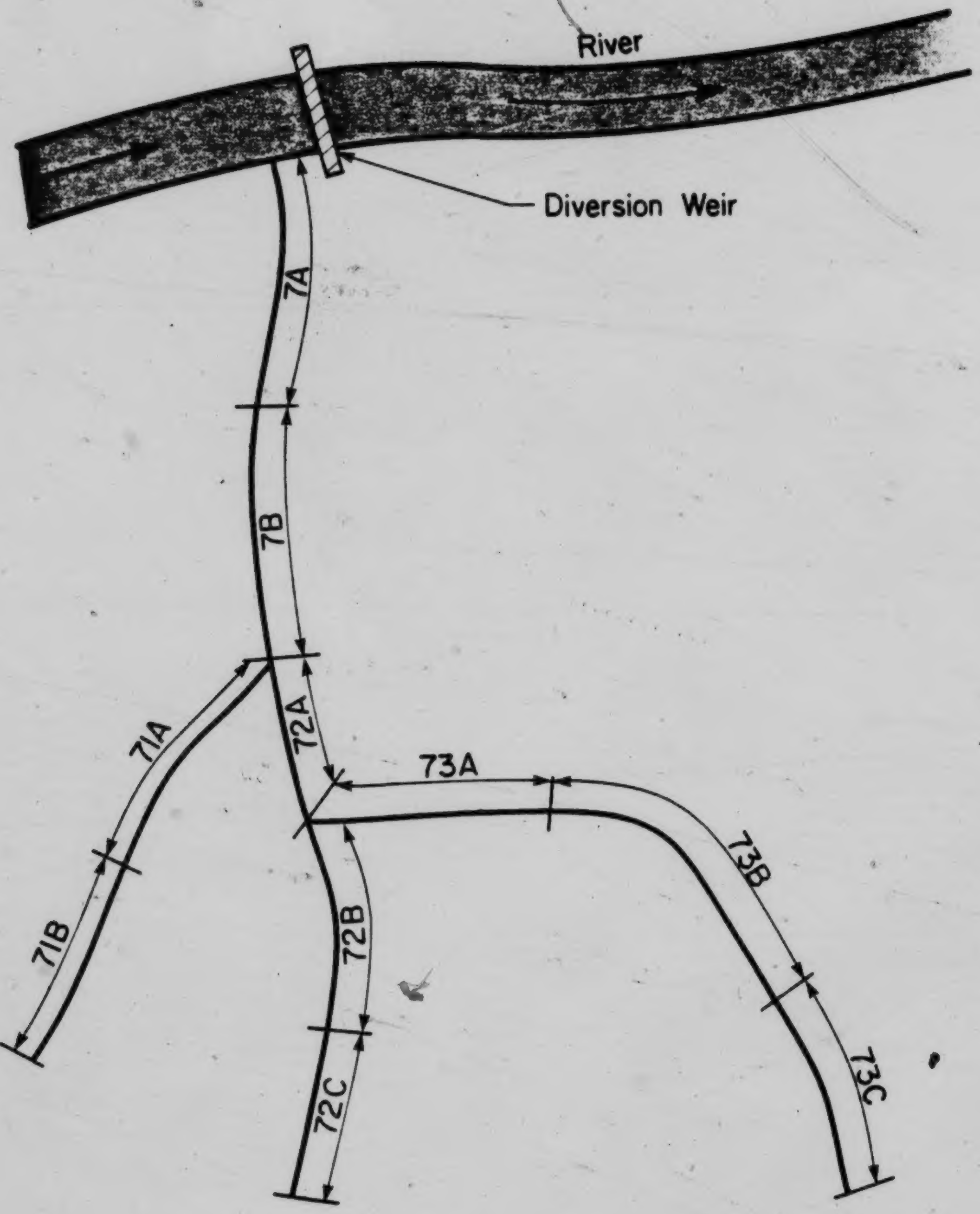

River
Diversion Weir
7A
7B
71A
71B
72A
72B
72C
73A
73B
73C

Figure C.1 Example of irrigation conveyance system divided into main, submains, and blocks.

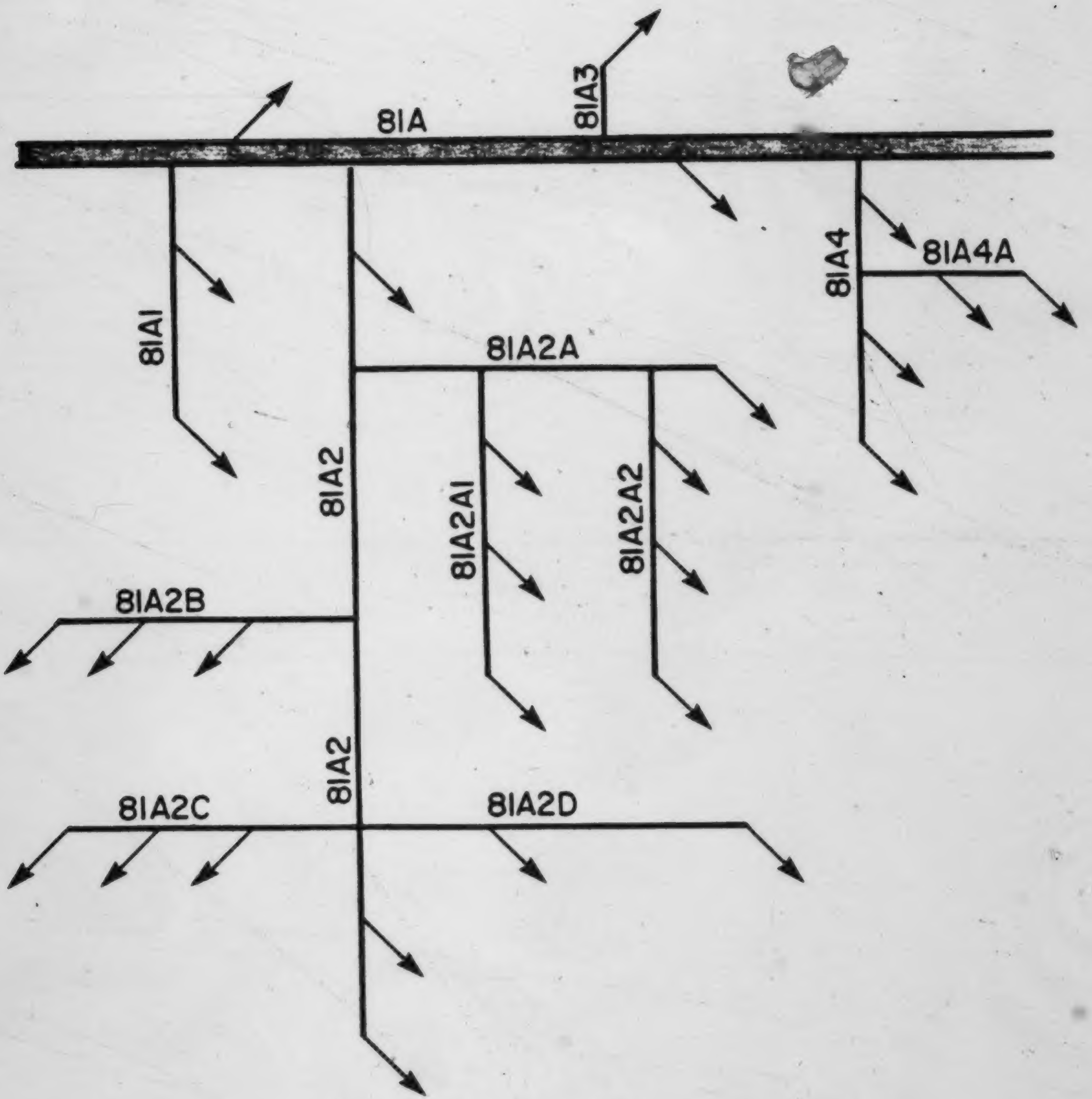

8IA
8IA3
8IA4
8IA4A
8IAI
8IA2A
8IA2
8IA2AI
8IA2A2
8IA2B
8IA2
8IA2C
8IA2D

Figure C.2 Example of alpha-numeric numbering system.

APPENDIX D

PHYSICAL CHARACTERISTICS OF STORAGE RESERVOIRS

APPENDIX D

PHYSICAL CHARACTERISTICS OF STORAGE RESERVOIRS

A schematic of a typical storage reservoir is shown in Figure D.1. Live storage is equal to the total water volume that can be withdrawn from the reservoir. Firm storage is equal to the water volume that can be withdrawn from the reservoir at the maximum outlet structure flow rate. Dead storage cannot be withdrawn from the reservoir at all. The maximum water stage permitted is called full supply level, fsl.

Flow through storage reservoirs may be described using the relation:

$$I - O = \frac{dS}{dt} \qquad \text{(D.1)}$$

where $I = \sum_{i}^{m} q_{inflow,i}$ (D.2)

$q_{inflow,i}$ = the inflow rate from source i;

$O = \sum_{i}^{k} q_{outflow,i}$ (D.3)

$q_{outflow,i}$ = the outflow rate at location i; and

S = storage in the reservoir usually expressed as a function of reservoir stage.

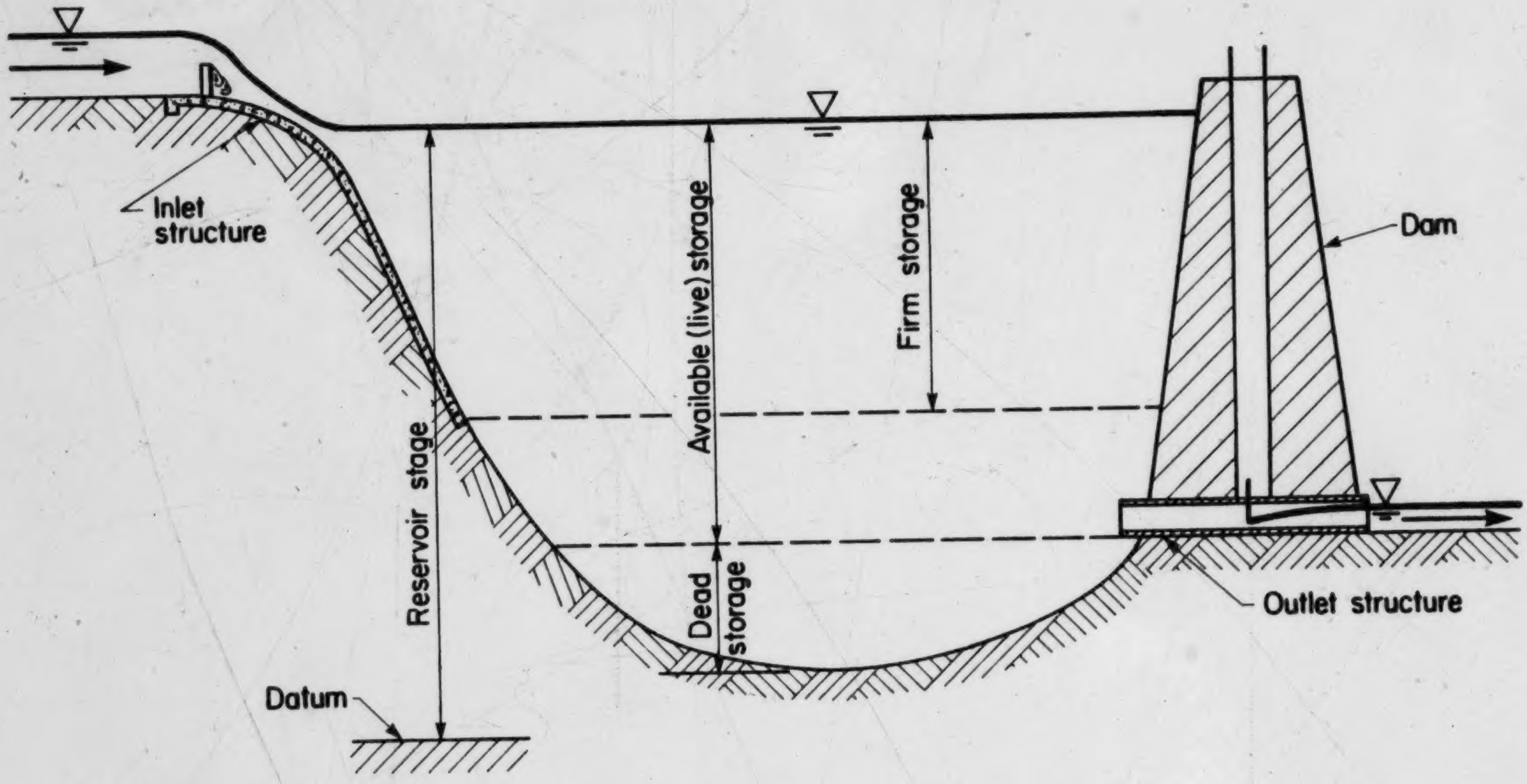

Inlet structure
Reservoir stage
Datum
Available (live) storage
Dead storage
Firm storage
Dam
Outlet structure

Figure D.1 Schematic of typical storage reservoir.

APPENDIX E

SEEPAGE FROM CANALS

APPENDIX E

SEEPAGE FROM CANALS

E.1 Introduction

Seepage is the process of water movement out of the canal through the canal wetted perimeter. Seepage is a complex function of several factors which include:

- the depth of water in the canal,
- the relationship between the canal and the local ground water,
- the length and shape of the canal wetted perimeter,
- the magnitude, homogenity and directional characteristics of the hydraulic conductivity of the soil material comprising the canal and the surrounding material over/through which the canal is constructed,
- the variability of ground water conditions; and
- the variability of canal flow conditions.

A very large body of literature exists which describes techniques capable of generating analytical solutions to most of the seepage problems encountered within irrigation conveyance systems. However, use of these techniques cannot normally be justified because of their exacting data requirements and the tremendous spatial and temporal variability of the pertinent physical parameters encountered in the field. If knowledge of seepage losses from a specific canal are required, seepage may be observed using one of a variety of techniques available for this purpose; Kraatz (1977), USBR (1976) and Kovda, et. al. (1973). If seepage losses cannot be measured directly, either because the canal doesn't exist as yet or the scope of the problem renders direct observation impractical, emperical methods must be resorted to.

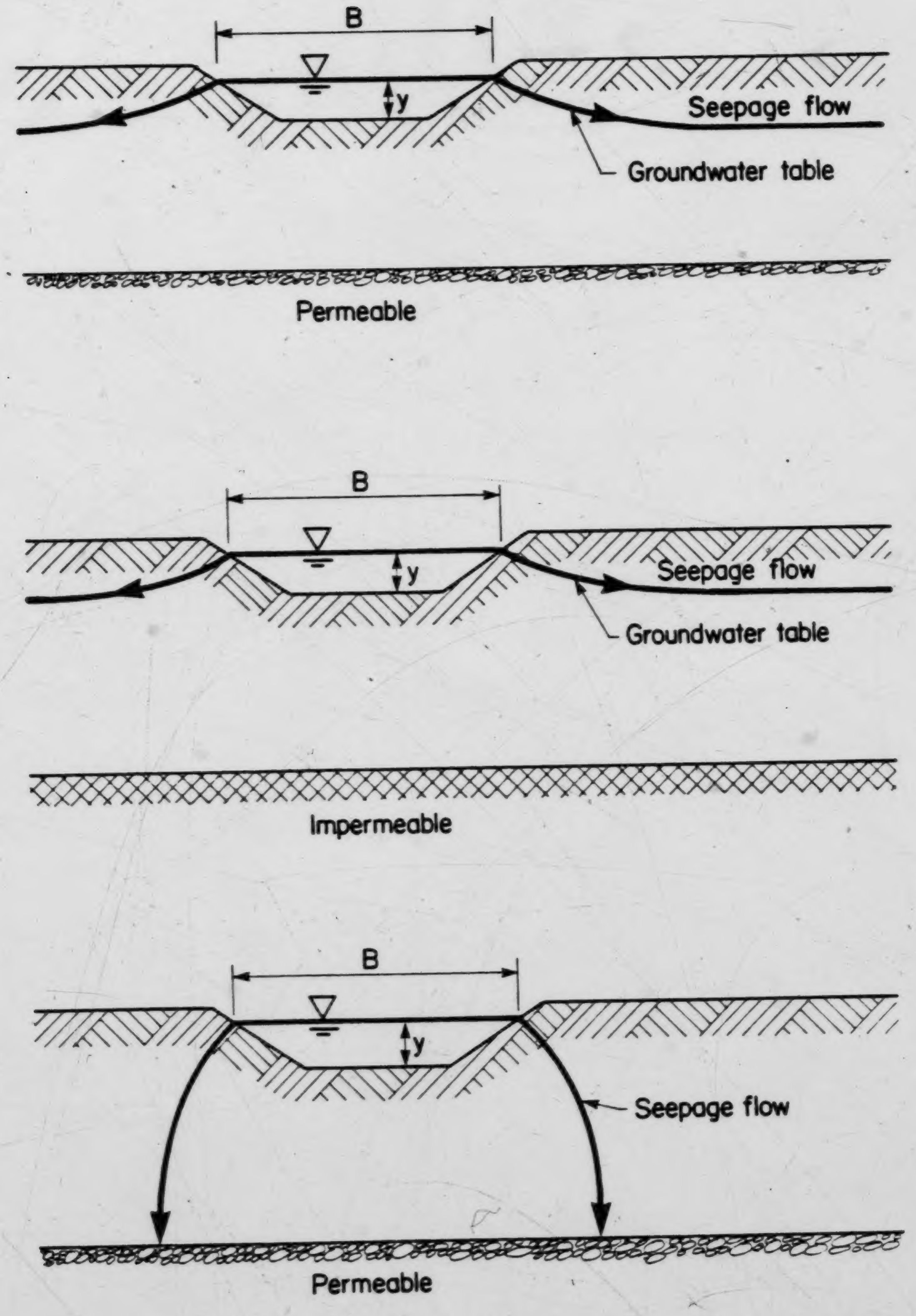

B
y
Seepage flow
Groundwater table
Permeable
B
y
Seepage flow
Groundwater table
Impermeable
B
y
Seepage flow
Permeable

Figure E.1 Typical seepage flow patterns from canals, Bouwer (1969).

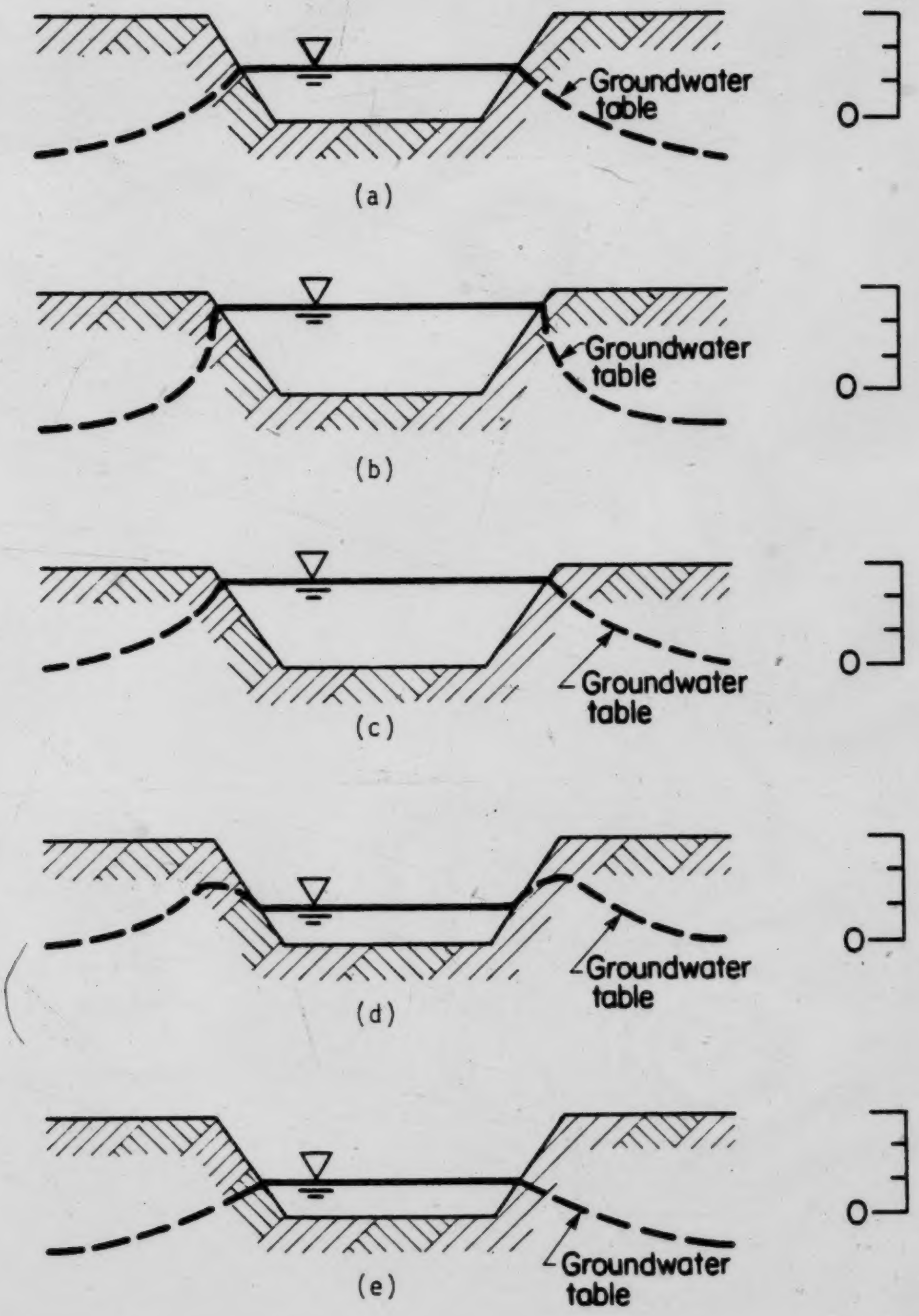

Groundwater table
0
(a)
Groundwater table
0
(b)
Groundwater table
0
(c)
Groundwater table
0
(d)
Groundwater table
0
(e)

Figure E.2 Seepage from canals flowing under unsteady conditions.

E.2 General Description of Seepage Problem

Typical steady-state flow patterns are described by Bouwer (1969) and are illustrated in Figure E.1. The three cases differ either with respect to the location of the surface of the groundwater table relative to the surface of the water in the canal or with respect to the permeability of the porous material underlying the strata of and through which the canal is constructed. Intuitively, if groundwater conditions remain constant, seepage losses will vary in direct response to changes in canal water depth. In Type A and B, variations in canal water depth would result in relatively similar variations in the hydraulic head causing the seepage to occur and in the seepage loss rates from the canal. In Type C the changes in canal water depth will not significantly change the hydraulic head causing seepage; however, seepage losses from the canal may vary in relation to variations in the canal wetted perimeter resulting from changes in canal water depth.

Analysis of seepage from canals under unsteady flow or groundwater conditions is extremely complex as is illustrated in Figure E.2. The relationship between the water level in the canal and the adjacent groundwater is initially steady-state as shown in Figure E.2(a). When the depth of water in the canal is suddenly increased, Figure E.2(b), increased seepage loss from the canal would result. These seepage losses would be expected to gradually decrease as the groundwater condition approached steady-state as shown in Figure E.2(c). If the water level in the canal was suddenly decreased to its original level, Figure E.2(d), drainage inflow to the canal could occur. The drainage inflow to the canal would gradually change to seepage outflow from the canal as the groundwater again approached steady-state conditions, Figure E.2(e). The phenomena just discussed is similar to the bankflow processes familiar to hydrologists studying hydrographs of flood flow in rivers. Linsley, Kohler and Paulhus, (1982). Mathematical analysis of these processes is presented by McWhorter and Sunada (1977).

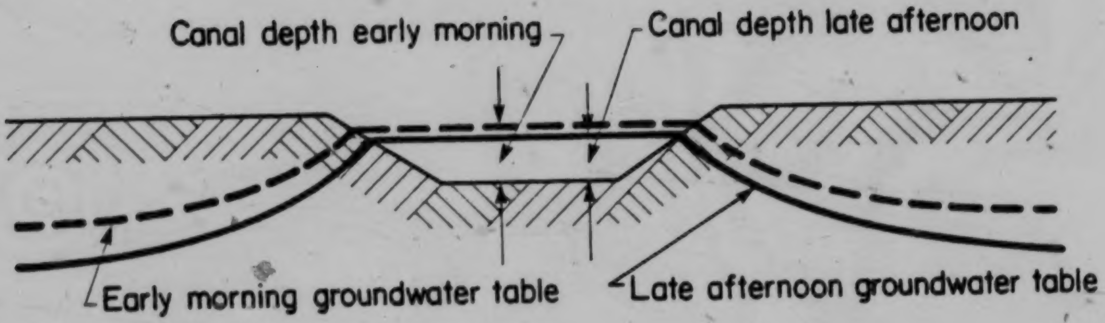
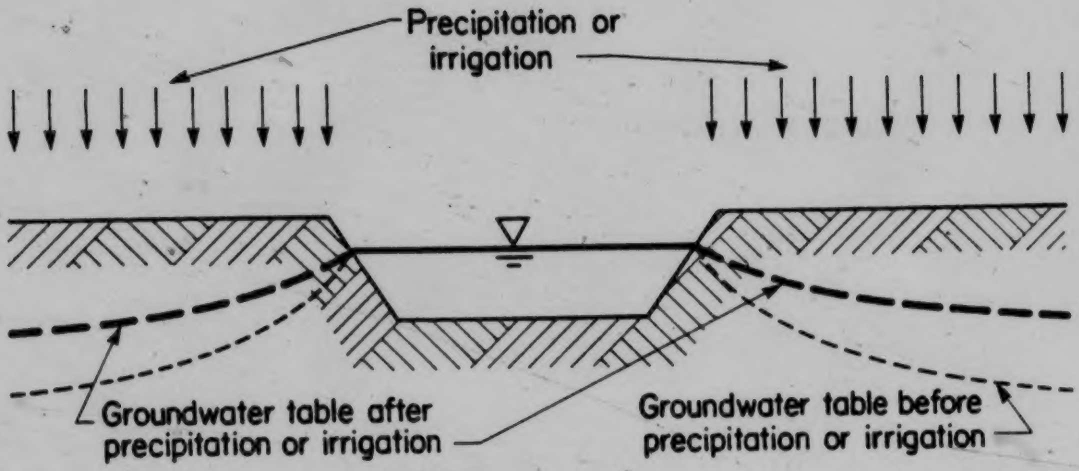

Canal depth early morning
Canal depth late afternoon
Early morning groundwater table
Late afternoon groundwater table

Figure E.3 Groundwater table with phreatophyte vegetation.

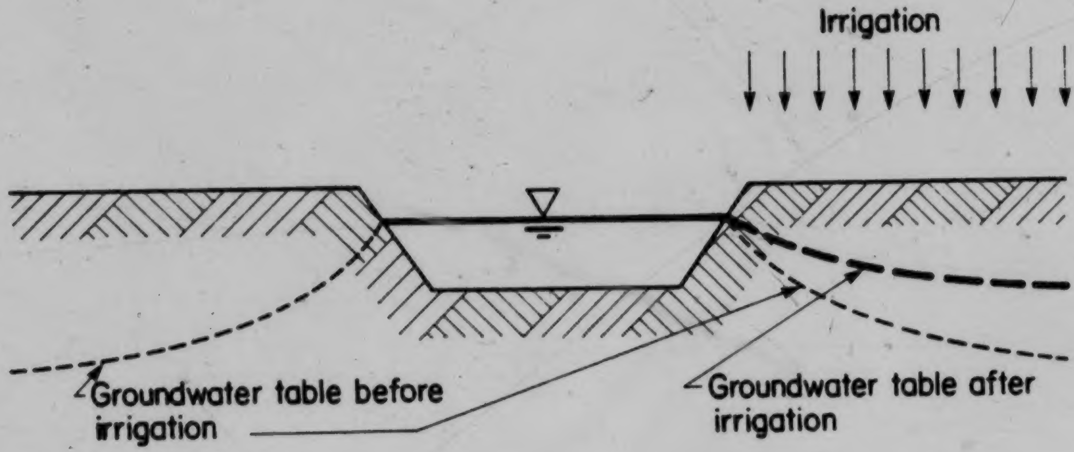
E.3 Phreatophyte Consumption of Water

Phreatophyte vegetation obtain most of their water from the saturated groundwater zone. Diurnal fluctuations of groundwater tables, which underlie areas overgrown with phreatophyte vegetation, have been documented and analyzed by many investigators including; White (1932), Truxell (1936), Thompson (1958), Blaney (1961), Meyboom (1967), van Hylckama (1970), Culler, et. al. (1976), Tromble (1977) and Burt (1979). Figure E.3 illustrates groundwater fluctuations near a Type A or Type B irrigation canal caused by phreatophyte water consumption. Meyboom (1967) and Burt (1979) have observed diurnal fluctuations in the discharges of small streams flowing through areas overgrown with phreatophyte vegetation. Both Meyboom and Burt noted that the discharge fluctuations were approximately in phase with potential evapotranspiration rates and attributed the fluctuations to phreatophyte water consumption.

Phreatophyte vegetation growing along irrigation canals can be expected to cause diurnal fluctuations in seepage losses. Though no actual observations (of diurnal seepage losses attributable to phreatophyte vegetation) have been reported in the literature, the author has had several such cases reported to him by canal operators employed by Irrigation Districts in Southern Alberta. Maximum seepage rates would be expected to occur in early afternoon and decrease to a minimum just before sunrise.


Precipitation or irrigation
Groundwater table after precipitation or irrigation
Groundwater table before precipitation or irrigation

(a) Precipitation or irrigation on both sides of the canal.


Irrigation
Groundwater table before irrigation
Groundwater table after irrigation

(b) Irrigation on one side of the canal.

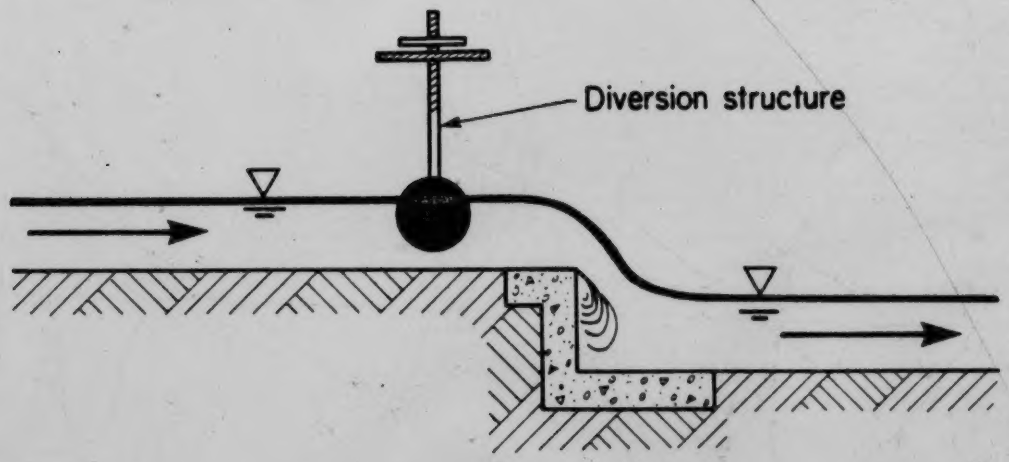
Figure E.4 Effect of precipitation and irrigation on groundwater and seepage.

E.4 Groundwater Recharge by Precipitation and Irrigation of Adjacent Fields

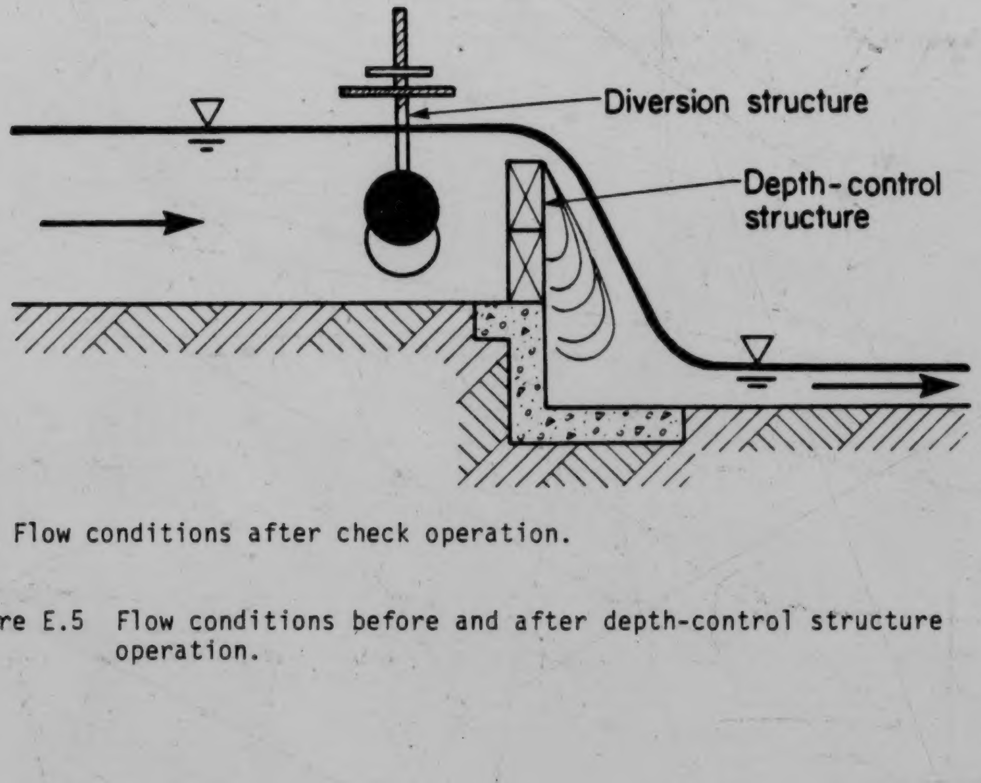
When land near Type A or Type B irrigation canals is irrigated or receives large amounts of precipitation, the groundwater next to the canal can be expected to be recharged. The effect of the recharge will be a reduction in the depth of the groundwater table and a subsequent reduction in seepage losses from the canal. These effects are illustrated in Figure E.4.

E.5 Temperature Effects

The temperature of the water and soil may affect the rate of seepage loss from irrigation canals. As the soil-water temperature increases during the summer months, the viscosity of the water will decrease and the hydraulic conductivity will increase. Significant seepage variation due to changes in seasonal soil-water temperatures have been observed in the Punjab, India by R.G. Kennedy as discussed by Etcheverry (1915(b)).


Diversion structure

(a) Flow conditions before check operation.


Diversion structure
Depth-control structure

(b) Flow conditions after check operation.

Figure E.5 Flow conditions before and after depth-control structure operation.

E.6 Operation of Depth-Control Structures

Depth-control structures are used to control water depth in canals. The main purpose of this control is to facilitate upstream water diversion activities as shown in Figure E.5. The resulting increase in water depth not only permits the operation of the diversion structure, but also, increases seepage losses from the canal as far upstream as the backwater effect extends. When the diversion operation is completed, the water depth is allowed to return to its pre-controlled conditions. Seepage losses due to the operation of depth-control structures cannot be avoided but they can be minimized by only increasing the water depth to the minimum required to make the diversion and by returning the depth-control structure to its pre-controlled condition as soon as the diversion is completed. In the vernacular of the "ditchriders" who operate the canals, "high-checking and leaving the boards in" are to be avoided.

E.7 Other Factors Which May Affect Seepage Losses from Irrigation Canals

Other factors which may affect seepage from irrigation canals include rodent activity, silt deposition or removal, the presence of roots of canal bank vegetation and the growth of aquatic and ditchbank weeds. Rodents, such as muskrats, ground squirrels and moles may burrow directly into the sides of canals. When the level of the water rises in the canal, significant and occasionally catestrophic water losses may occur. Similar difficulties are presented by the roots of canal bank vegetation which upon dying may leave relatively large continuous paths for water to escape the canal. Canal seepage losses may be reduced if silt is deposited; however, this benefit is normally gained only after several irrigation seasons and is essentially lost with every maintenance cycle. Aquatic and ditchbank weeds will increase the resistance to water flow in the canal. As the extent of the weed growth increases, greater depths of water are required to pass the same discharge. Accompanying the increase in depth is increased seepage. The impact on seepage losses from irrigation canals by rodent activity, silt deposition, the presence of roots and the growth of acquatic and ditchbank weeds is also a function of the frequency and quality of canal maintenance directed toward controlling these factors.

E.8 Estimation of Seepage Losses Using Emperical Methods

Several emperical techniques have been developed to estimate seepage losses from irrigation canals. Five of the techniques, chosen because of their potential for local application, are presented.

E.8.1 Moritz Formula

This method was originally proposed by Moritz (1913) and is extensively discussed by Etcheverry (1915(b)), Kovda, et. al (1973) and Kraatz (1977). The Moritz formula was adopted for use by the United States Bureau of Reclamation, USBR (1967), and the Alberta Department of Agriculture Irrigation Division (1979).

Seepage rates are estimated using the following formula:

$$q_s = 1.16 \times 10^{-5} C_m A^{\frac{1}{2}} \qquad (E.1)$$

where q_s is the seepage rate measured in cubic meters per second per meter of canal length;

A is the cross-sectional area of the canal measured in square meters; and

C_m is a coefficient which is a function of the material used to construct the canal. These coefficients may be obtained from Table E.1, (from Kraatz (1977)).

Table E.1 Moritz Coefficients

Soil Type	C_m
Cemented gravel and hardpan with sandy loam	0.34
Clay and clayey loam	0.41
Sandy loam	0.66
Volcanic ash	0.68
Sand and volcanic ash or clay	1.20
Sandy soil with rock	1.68
Sandy and gravelly soil	2.20

E.8.2 India Formula

This method for estimating seepage is discussed in ICID (1967) and employs the following equation:

$$q_s = 1.923 \times 10^{-6} \cdot Q^{0.0652} \cdot P \qquad \text{(E.2)}$$

where q_s is the seepage rate in cubic meters per second per meter of canal length;

Q is the rate of flow in the canal in cubic meters per second; and,

P is the wetted perimeter of the canal in meters.

An alternate formula used to estimate seepage from a different group of canals in India was presented as well. The equation is as follows:

$$q_s = C_I \, P \cdot d \times 10^{-6} \qquad \text{(E.3)}$$

where d is the depth of flow in meters

and C_I is a constant which varies from 1.1 to 1.8.

The remaining variables are defined as before. Unfortunately, no guidelines are presented for the selection of C_I. This method is also discussed in Kraatz (1977). It is worth noting that a formula similar to Equation E.2 is used in Pakistan to estimate seepage losses from canals.

E.8.3 Egypt Formula

This method for estimating seepage is presented in ICID (1967) and again in Kraatz (1977). The formula is as follows:

$$q_s = C_E \, P \, y_r^{0.5} \times 10^{-3} \qquad \text{(E.4)}$$

where q_s is the seepage rate in cubic meters per second per meter length of canal;

P is the wetted perimeter;

y_r is the hydraulic mean depth which is equal to the cross-sectional area of the canal divided by its top width; and,

C_E is a coefficient which varies from 0.0015 for stiff clay soils to 0.0030 for sandy soils.

E.8.4 USSR Formula

This method for estimating seepage is discussed in ICID (1967), Kraatz (1977) and Kovda, et. al. (1973) and is used when specific information about the groundwater is unavailable. The formula is as follows:

$$q_s = C_U \, Q^{(1-m)} \times 10^{-5} \qquad \text{(E.5)}$$

where q_s is the seepage rate in cubic meters per second per meter of canal length; and,

C_U and m are constants which are a function of canal bank permeability. Typical values are presented in Table E.2 (from Kovda, et. al. (1973)).

Table E.2 Values of C_U and m

	Permeability		
	low	medium	high
C_U	0.70	1.90	3.40
m	0.30	0.40	0.50

E.8.5 Akram, Kemper, Sabey Formula

This formula was developed during seepage loss studies performed in Pakistan and Colorado, Akram, et. al. (1981). The formula proposed is as follows:

$$q_s = a \, e^{-b(y_{op}-y)} \qquad \text{(E.6)}$$

where q_s is the rate of seepage in cubic meters per second per meter length of canal;

y_{op} is the actual depth of water in the canal in meters, when it is operating at maximum rate of flow;

y is the actual depth of water in the canal, in meters, at rates of flow other than maximum; and,

a and b are constants whose values are specific to the canal
being studied.

The authors of this formula found that Equation E.6 fit observed seepage loss rates very well.

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APPENDIX F

EVAPORATION OF WATER FROM IRRIGATION CANALS AND STORAGE FACILITIES

APPENDIX F

EVAPORATION OF WATER FROM IRRIGATION CANALS AND STORAGE FACILITIES

Evaporation of water will take place from the surfaces of all canals and storage facilities found within irrigation conveyance systems. Evaporation water losses from these water surfaces is not significantly affected, either positively or negatively, by the presence of emergent or free-floating aquatic vegetation, Idso (1981). Though evaporation losses from storage facilities, which have a relatively large surface area to volume ratio, may be considerable, evaporation losses from flowing canals are normally considered negligible and may be neglected, particularly in water balance studies of daily or hourly duration. This is, of course, not the case in ponding studies where seepage loss rates from canals are being determined, Kraatz (1977). Formulae useful for estimating evaporation from open water surfaces may be found in several standard hydrology references including; Linsley, et. al. (1982), Gray (1970), Viessman, et. al. (1977), Wilson (1974), Eagleson (1970), Chow (1964) and Gangopadhyaya, et. al. (1966). An equation developed specifically for estimating evapotranspiration from vegetated water surfaces was developed by Eisenlohr (1966). This equation does not significantly differ in form from those used to estimate water losses from non-vegetated or open-water surfaces.

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APPENDIX G

DIRECT PRECIPITATION INFLOW TO CANALS AND STORAGE FACILITIES

APPENDIX G

DIRECT PRECIPITATION INFLOW TO CANALS AND STORAGE FACILITIES

Direct precipitation inflow to canals and storage facilities represents an uncontrolled and potentially hazardous water input to the irrigation conveyance system. Direct precipitation inflow should be distinguished from drainage inflow which is precipitation that falls on the adjacent land areas outside of the canal and reservoir area and drains into them.

The rate of direct precipitation inflow to canals may be estimated using the following equation:

$$p = 2.778 \times 10^{-6} I_p A_c \qquad \text{(G.1)}$$

where p is the precipitation inflow to the canal in cubic meters per second per meter of canal length,

I_p is the intensity of precipitation measured in centimeters per hour; and,

A_c is the projected surface area of the canal per unit length of canal measured in square meters per meter. See Figure G.1.

The rate of direct precipitation inflow to storage facilities may be somewhat more difficult to determine but may be estimated using the relation:

$$Q_p = 2.778 \times 10^{-6} I_p A_{FSL} \qquad \text{(G.2)}$$

where Q_p is the total rate of direct precipitation inflow to the storage facility; and,

A_{FSL} is the surface area of the storage facility at full supply level in square meters.

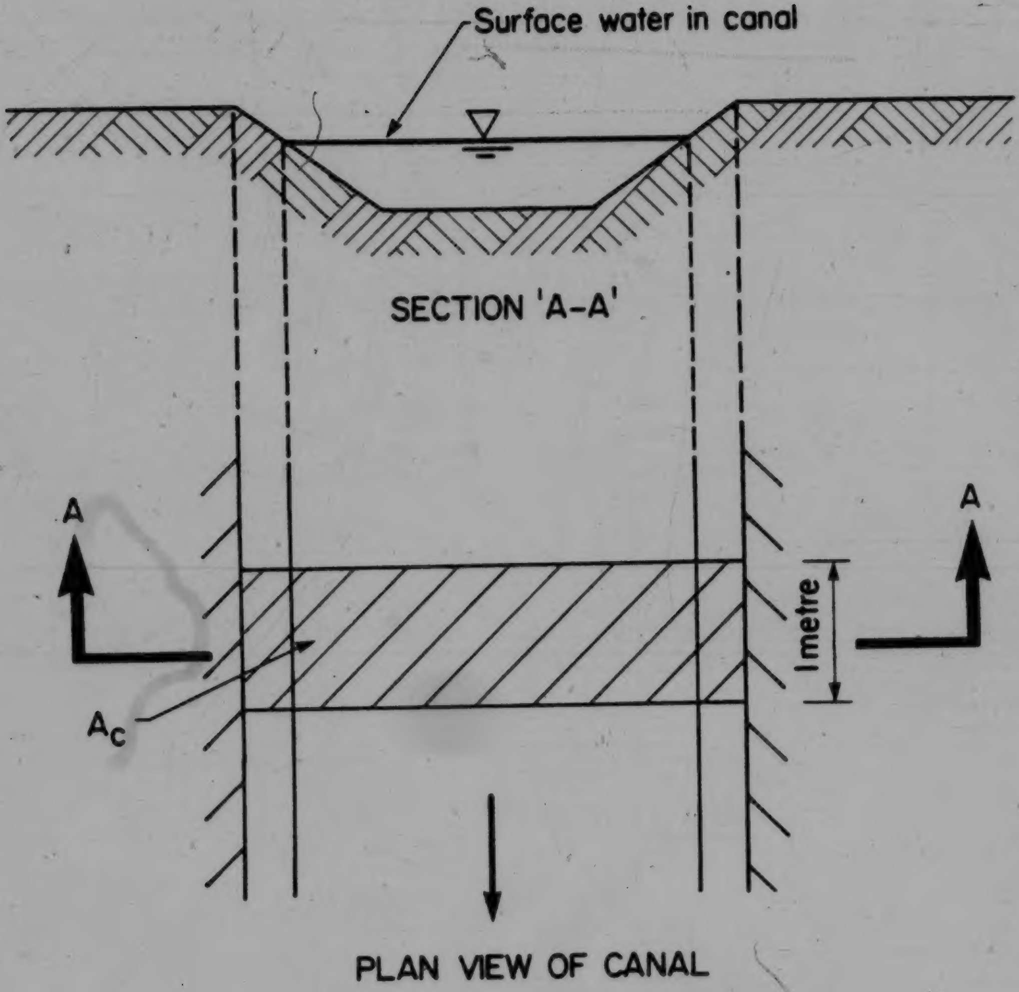
Surface water in canal
SECTION 'A-A'
A
A
A$_c$
I metre
PLAN VIEW OF CANAL

Figure G.1 Definition sketch showing projected area of canal.

APPENDIX H

LISTING OF THE
IRRIGATION CONVEYANCE SYSTEM
SIMULATION MODEL

```
      PROGRAM ICSS1
C     ******VERSION 1******
C
C
C
C
C
C     THIS FIRST SECTION OF THE PROGRAM CONTROLS THE INTERACTION
C     BETWEEN THE VARIOUS SUBROUTINES AND RETRIEVES THE INPUT DATA
C     FROM FILE 'SIMDAT1' AND 'RCHDAT1' CREATED BY THE SEPARATE
C     PROGRAMS 'SIMDAT' AND 'RCHDAT' REPECTIVELY.
C
      COMMON /CKRCH/KRCH
      COMMON /CTTHR/TTHR
      COMMON /CNREACH/NREACH
      COMMON /CCTTHR/CTTHR
      COMMON /CTDAY/TDAY
      COMMON /CCMBAL/CMBAL(10,20)
      OPEN (UNIT=1,FILE='SIM1',STATUS='OLD')
      OPEN (UNIT=2,FILE='RCH1',STATUS='OLD')
      OPEN (UNIT=3,FILE='DATAM1',STATUS='NEW')
      OPEN (UNIT=4,FILE='BACKW1',STATUS='NEW')
      OPEN (UNIT=5,FILE='REPORT1',STATUS='NEW')

      OPEN (UNIT=6,FILE='REPORT2',STATUS='NEW')
      OPEN (UNIT=7,FILE='REPORT3',STATUS='NEW')
      OPEN (UNIT=8,FILE='REPORT4',STATUS='NEW')
      OPEN (UNIT=9,FILE='REPORT5',STATUS='NEW')
      OPEN (UNIT=10,FILE='REPORT6',STATUS='NEW')
      OPEN (UNIT=11,FILE='OPERAT1',STATUS='NEW')
      OPEN (UNIT=12,FILE='DBCPAR',STATUS='NEW')
      OPEN (UNIT=13,FILE='UBCPAR',STATUS='NEW')
      READ (1,1000) TSTUD
 1000 FORMAT (1X,F8.4)
      READ (1,950) TDAY
  950 FORMAT (1X,F6.2)
      READ (1,900) THYINH
  900 FORMAT (1X,F8.4)
      READ (1,850) THYDRO
  850 FORMAT (1X,F8.4)
      READ (1,800) NREACH
  800 FORMAT (1X,I2)
      READ (1,700) WFACTA
  700 FORMAT (1X,F4.2)
      READ (1,650) WFACTB
  650 FORMAT (1X,F4.2)
      READ (1,600) RESMAX
  600 FORMAT (1X,F5.4)
      READ (1,500) GRAV
  500 FORMAT (1X,F10.8)
      DUM1=THYDRO/THYINH
      NHYDIN=IFIX(DUM1)
      THYINS=THYINH*3600.
      CALL DATAM(NREACH,THYINS)
C     INITIALIZATION OF CUMMULATIVE WATER BALANCE TERMS
      DO 18 I2=1,NREACH
      DO 17 I=1,20
      CMBAL(I2,I)=0.0
   17 CONTINUE
   18 CONTINUE
      CALL BACKW(NREACH,GRAV)
      CALL COCALC(NREACH)
      TTHR=0.0
      CTTHR=0.0
      DO 15 KD=1,NREACH
      KRCH=KD
      CALL REPORT(THYINS)
   15 CONTINUE
   10 CONTINUE
      IF (CTTHR.GE.TSTUD) GO TO 20

      CALL OPERAT(TNOP)
      DUM2=TNOP/THYDRO+1.
      NHYROI=IFIX(DUM2)
      TDNOP=FLOAT(NHYROI)*THYDRO
      CTTHR=CTTHR+TDNOP
      DO 30 K1=1,NHYROI
      DO 35 KRCH=1,NREACH
      CALL HYDROL
   35 CONTINUE
      DO 40 K2=1,NHYDIN
      TTHR=TTHR+THYINH
      DO 50 KRCH=1,NREACH
      CALL HYDRT(GRAV,WFACTA,WFACTB,THYINS,RESMAX)
      CALL BCPREP
      CALL REPORT(THYINS)
   50 CONTINUE
   40 CONTINUE
   30 CONTINUE
      GO TO 10
   20 CONTINUE
      PRINT 1001
 1001 FORMAT(/,1X,'DURATION OF STUDY IS NOW EXCEEDED.')
      PRINT 1002
 1002 FORMAT(1X,'IF YOU WISH TO TERMINATE THE STUDY, INPUT "999.0".')
      PRINT 1003
 1003 FORMAT(1X,'IF YOU WISH TO CONTINUE THE STUDY, INPUT THE NO. OF')
      PRINT 1004
 1004 FORMAT(1X,'HOURS YOU WISH THE STUDY TO CONTINUE.')
      READ *,EXHOUR
      IF (EXHOUR.GT.50.0) GO TO 60
      TSTUD=TSTUD+EXHOUR
      GO TO 10
   60 CONTINUE
      PRINT 1997
 1997 FORMAT (/,1X,'PROGRAM TERMINATED.')
      END
C
C
C
C
C
      SUBROUTINE DATAM(NREACH,THYINS)
C
C     THIS SUBROUTINE CREATES A FILE NAMED 'DATAM1', IN WHICH
C     ALL INPUT VALUES FROM 'RCHDAT' FOR EACH REACH ARE IDENTIFIED
C     WITH THEIR APPROPRIATE VARIABLE DESCRIPTION.

C
      COMMON /CRCHPR/RCHPR(10,5)
      COMMON /CICHID/ICHID(10,10)
      COMMON /CELEV/ELEV(10,21)
      COMMON /CGEOM/GEOM(10,21)
      COMMON /CROUGH/ROUGH(10,20)
      COMMON /CQLAT/QLAT(10)
      COMMON /CQSEEP/QSEEP(10,20)
      COMMON /CQPREC/QPREC(10,20)
      COMMON /CQODST/QODST(10,20)
      COMMON /CQEVAP/QEVAP(10,20)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CIREP/IREP(10,12)
      READ (2,1999) NREACH
 1999 FORMAT (1X,I2)
      N=0
      DO 70 K3=1,NREACH
      N=N+1
      READ(2,2000) KRCH
 2000 FORMAT (1X,I2)
      ICHID(KRCH,1)=KRCH
      READ(2,2001) RCHPR(KRCH,1),RCHPR(KRCH,2),RCHPR(KRCH,5)
```

```
2001 FORMAT (1X,F8.2,2F6.2)
     RCHPR(KRCH,3)=RCHPR(KRCH,1)/(RCHPR(KRCH,2)-1)
     RCHPR(KRCH,4)=THYINS/RCHPR(KRCH,3)
     DUM1=RCHPR(KRCH,2)+0.5
     K4=IFIX(DUM1)
     READ(2,2002) ELEV(KRCH,1),ELEV(KRCH,K4)
2002 FORMAT (1X,2F8.2)
     K5=K4-1
     DELEV=(ELEV(KRCH,1)-ELEV(KRCH,K4))/(RCHPR(KRCH,2)-1)
     R1=0.0
     DO 80 K6=2 , K5
     R1=R1+1.0
     ELEV(KRCH,K6)=ELEV(KRCH,1)-DELEV*R1
  80 CONTINUE
     READ(2,2003) (GEOM(KRCH,K7),K7=1,12)
2003 FORMAT (1X,F8.4,2X,F8.4,2X,F8.4,2X,F8.4,2X)
     READ(2,2027) (GEOM(KRCH,K7),K7=13,21)
2027 FORMAT (1X,F8.4,2X,F8.4,2X,F8.4,2X)
     READ(2,2004) ICHID(KRCH,2)
2004 FORMAT (1X,I2)
     READ(2,2005) ROUGH(KRCH,1)
2005 FORMAT (1X,F4.0)
     DUM2=ROUGH(KRCH,1)+0.5

     K8=IFIX(DUM2)+1
     READ(2,2006) (ROUGH(KRCH,K9), K9=2,K8)
2006 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
     READ(2,2007) QLAT(KRCH)
2007 FORMAT (1X,F12.6)
     READ(2,2008) ICHID(KRCH,3)
2008 FORMAT (1X,I2)
     READ(2,2009) ICHID(KRCH,4)
2009 FORMAT (1X,I2)
     READ(2,2010) QSEEP(KRCH,1)
2010 FORMAT (1X,F4.0)
     DUM3=QSEEP(KRCH,1)+0.5
     K10=IFIX(DUM3)+1
     READ(2,2011) (QSEEP(KRCH,K11),K11=2,K10)
2011 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
     READ(2,2012) ICHID(KRCH,5)
2012 FORMAT (1X,I2)
     READ(2,2013) QPREC(KRCH,1)
2013 FORMAT (1X,F4.0)
     DUM4=QPREC(KRCH,1)+0.5
     K12=IFIX(DUM4)+1
     READ(2,2014) (QPREC(KRCH,K13),K13=2,K12)
2014 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
```

```
      READ(2,2015) ICHID(KRCH,6)
 2015 FORMAT (1X,I2)
      READ(2,2016) QEVAP(KRCH,1)
 2016 FORMAT (1X,F4.0)
      DUM5=QEVAP(KRCH,1)+0.5
      K14=IFIX(DUM5)+1
      READ(2,2017) (QEVAP(KRCH,K15),K15=2,K14)
 2017 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      READ(2,2018) ICHID(KRCH,7)
 2018 FORMAT (1X,I2)
      READ(2,2019) QODST(KRCH,1)
 2019 FORMAT (1X,F4.0)
      DUM6=QODST(KRCH,1)+0.5
      K16=IFIX(DUM6)+1
      READ(2,2020) (QODST(KRCH,K17),K17=2,K16)
 2020 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      READ(2,2021) ICHID(KRCH,8)
 2021 FORMAT(1X,I2)
      READ(2,2021) ICHID(KRCH,9)
      READ(2,2022) UBCPR(KRCH,1)
 2022 FORMAT(1X,F4.0)
      DUM7=UBCPR(KRCH,1)+0.5
      K18=IFIX(DUM7)+1

      READ(2,2023) (UBCPR(KRCH,K19),K19=2,K18)
 2023 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      READ(2,2024) DBCPR(KRCH,1)
 2024 FORMAT (1X,F4.0)
      DUM8=DBCPR(KRCH,1)+0.5
      K20=IFIX(DUM8)+1
      READ(2,2025) (DBCPR(KRCH,K21),K21=2,K20)
 2025 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      READ(2,2026) (IREP(KRCH,K22),K22=1,12)
 2026 FORMAT (1X,12I2)
      IF(N.GT.1) GO TO 95
      WRITE(3,2129) NREACH
 2129 FORMAT (1X,'TOTAL NUMBER OF REACHES: ',I2)
      WRITE(3,2130)
 2130 FORMAT (1X,'-----------------------------')
   95 CONTINUE
      I5=K3
      IF(IREP(I5,1).EQ.2) GO TO 70
      WRITE(3,2100)
 2100 FORMAT (1X,'REACH NUMBER')
      WRITE(3,2101) I5
 2101 FORMAT (1X,I2)
      WRITE(3,2102)
```

```
2102 FORMAT (1X,'REACH INDICES')
     WRITE(3,2103) (ICHID(I5,I),I=1,9)
2103 FORMAT (1X,9(I2,5X))
     WRITE(3,2104)
2104 FORMAT (1X,'REACH PARAMETERS')
     WRITE(3,2105) (RCHPR(I5,I),I=1,5)
2105 FORMAT (1X,F8.2,5X,F6.2,5X,F6.2,5X,F8.4,5X,F6.2)
     WRITE(3,2106)
2106 FORMAT (1X,'ELEVATION OF REACH BOTTOM')
     WRITE(3,2107)(ELEV(I5,I),I=1,K4)
2107 FORMAT (1X,F10.4,5X,F10.4,5X,F10.4,5X,F10.4,5X,F10.4,5X,F10.4,5X)
     WRITE(3,2108)
2108 FORMAT (1X,'CHANNEL GEOMETRY COEFFICIENTS')
     WRITE(3,2109) (GEOM(I5,I),I=1,12)
2109 FORMAT (1X,F8.4,5X,F8.4,5X,F8.4,5X,F8.4,5X)
     WRITE(3,2131) (GEOM(I5,I),I=13,21)
2131 FORMAT (1X,F8.4,5X,F8.4,5X,F8.4,5X)
     WRITE(3,2110)
2110 FORMAT (1X,'ROUGHNESS PARAMETERS')
     WRITE(3,2111) (ROUGH(I5,I),I=1,K8)
2111 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2112)
2112 FORMAT (1X,'TOTAL LATERAL FLOW---NEGATIVE IF INFLOW')

     WRITE(3,2113) QLAT(I5)
2113 FORMAT (1X,F10.8)
     WRITE(3,2114)
2114 FORMAT (1X,'SEEPAGE PARAMETERS')
     WRITE(3,2115) (QSEEP(I5,I),I=1,K10)
2115 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2116)
2116 FORMAT (1X,'PRECIPITATION PARAMETERS')
     WRITE(3,2117) (QPREC(I5,I),I=1,K12)
2117 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2118)
2118 FORMAT (1X,'EVAPORATION PARAMETERS')
     WRITE(3,2119) (QEVAP(I5,I),I=1,K14)
2119 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2120)
2120 FORMAT (1X,'OTHER DISTRIBUTED INFLOW PARAMETERS')
     WRITE(3,2121) (QODST(I5,I),I=1,K16)
2121 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2122)
2122 FORMAT (1X,'UPSTREAM BOUNDARY CONDITION PARAMETERS')
     WRITE(3,2123) (UBCPR(I5,I),I=1,K18)
2123 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
     WRITE(3,2124)
```

```
 2124 FORMAT (1X,'DOWNSTREAM BOUNDARY CONDITION PARAMETERS')
      WRITE(3,2125) (DBCPR(I5,I),I=1,K20)
 2125 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
      WRITE(3,2126)
 2126 FORMAT (1X,'REPORT CONTROL INDICES')
      WRITE(3,2127) (IREP(I5,I),I=1,12)
 2127 FORMAT (1X,12(I2,5X))
      WRITE(3,2128)
 2128 FORMAT (1X,'****************************************************')
   70 CONTINUE
      RETURN
      END
C
C
C
C
C
C     THE FOLLOWING SINGLE PRECISION FUNCTION ROUTINES CALCULATE
C     THE AREA, TOP WIDTH, WETTED PERIMETER, DA/DY, DB/DY AND
C     DP/DY VALUES USING THE GEOM MATRIX CREATED BY THE PROGRAM
C     RCHDAT. THESE ROUTINES ARE EXECUTED WHEN THE NAME APPEARS
C     IN THE CALLING PROGRAM UNIT. (EG. SUBROUTINE 'BACKW'.)
      FUNCTION AREA(DEPTH)

C
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      AREA=GEOM(KRCH,1)+GEOM(KRCH,2)*DEPTH+GEOM(KRCH,3)*DEPTH*DEPTH+GEOM
     1(KRCH,4)*DEPTH*DEPTH*DEPTH
      RETURN
      END
C
      FUNCTION TOPW(DEPTH)
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      TOPW=GEOM(KRCH,5)+GEOM(KRCH,6)*DEPTH+GEOM(KRCH,7)*DEPTH*DEPTH+GEOM
     1(KRCH,8)*DEPTH*DEPTH*DEPTH
      RETURN
      END
C
      FUNCTION WETP(DEPTH)
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      WETP=GEOM(KRCH,9)+GEOM(KRCH,10)*DEPTH+GEOM(KRCH,11)*DEPTH*DEPTH+GE
     1OM(KRCH,12)*DEPTH*DEPTH*DEPTH
      RETURN
      END
```

```
C
      FUNCTION DAREA(DEPTH)
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      DAREA=GEOM(KRCH,13)+GEOM(KRCH,14)*DEPTH+GEOM(KRCH,15)*DEPTH*DEPTH
      RETURN
      END
C
      FUNCTION DTOPW(DEPTH)
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      DTOPW=GEOM(KRCH,16)+GEOM(KRCH,17)*DEPTH+GEOM(KRCH,18)*DEPTH*DEPTH
      RETURN
      END
C
      FUNCTION DWETP(DEPTH)
      COMMON /CKRCH/KRCH
      COMMON /CGEOM/GEOM(10,21)
      DWETP=GEOM(KRCH,19)+GEOM(KRCH,20)*DEPTH+GEOM(KRCH,21)*DEPTH*DEPTH
      RETURN
      END
C
C

C
C
C
      SUBROUTINE BACKW(NREACH,GRAV)
C
C     THIS SUBROUTINE PERFORMS THE BACKWATER CALCULATIONS AND
C     CREATES A DATA FILE NAMED 'BACKW1', IN WHICH THE FOLLOWING
C     PARAMETERS ARE STORED: REACH NUMBER, LENGTH OF REACH, NUMBER
C     OF NODES, DISTANCE INCREMENT BETWEEN NODES AND
C     PER NODE: ELEVATION, DEPTH AND VELOCITY.
C
      DIMENSION QRCH(21)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CDEPTH/DEPTH(10,21)
      COMMON /CVEL/VEL(10,21)
      COMMON /CRCHPR/RCHPR(10,5)
      COMMON /CQLAT/QLAT(10)
      COMMON /CKRCH/KRCH
      COMMON /CIREP/IREP(10,12)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CELEV/ELEV(10,21)
      COMMON /CROUGH/ROUGH(10,20)
      DO 100 KR=1,NREACH
      KRCH=(NREACH+1)-KR
      DUM1=RCHPR(KRCH,2)+0.5
      I=IFIX(DUM1)
      DEPTH(KRCH,I)=DBCPR(KRCH,5)
      QRCH(I)=DBCPR(KRCH,4)
      VEL(KRCH,I)=QRCH(I)/AREA(DEPTH(KRCH,I))
      I1=I-1
      N12=0
  444 CONTINUE
      IF(N12.GT.20) GO TO 4005
      Q=-RCHPR(KRCH,3)*QLAT(KRCH)
      DO 110 I2=1,I1
      I3=I-I2
      I4=I3+1
      DEPTH(KRCH,I3)=DEPTH(KRCH,I4)
      QRCH(I3)=QRCH(I4)+Q
      A1=AREA(DEPTH(KRCH,I4))
      B1=TOPW(DEPTH(KRCH,I4))
      P1=WETP(DEPTH(KRCH,I4))
      N11=0
  120 CONTINUE

      IF(N11.GT.20) GO TO 4000
      DP=DWETP(DEPTH(KRCH,I3))
      A=AREA(DEPTH(KRCH,I3))
      B=TOPW(DEPTH(KRCH,I3))
      P=WETP(DEPTH(KRCH,I3))
      N11=N11+1
      F=DEPTH(KRCH,I4)-DEPTH(KRCH,I3)+RCHPR(KRCH,3)/2.0/GRAV*QLAT(KRCH)*
     1(QRCH(I4)/A1/A1+QRCH(I3)/A/A)-(QRCH(I4)*QRCH(I4)/2.0/GRAV*(1.0/A1/
     2A1-A/A1/A1/A1))-(QRCH(I3)*QRCH(I3)/2.0/GRAV*(A1/A/A/A-1.0/A/A))+EL
     3EV(KRCH,I4)-ELEV(KRCH,I3)+RCHPR(KRCH,3)*ROUGH(KRCH,2)*ROUGH(KRCH,2
     4)/2.0*((QRCH(I4)*QRCH(I4)*(P1**1.33333)/(A1**3.33333))+(QRCH(I3)*Q
     5RCH(I3)*(P**1.33333)/(A**3.33333)))
      F=-F
      DF=1.0+(RCHPR(KRCH,3)/GRAV*QLAT(KRCH)*QRCH(I3)*B/A/A/A)-(1.5/GRAV*
     1QRCH(I3)*QRCH(I3)*B*A1/A/A/A/A)-(0.5/GRAV*QRCH(I4)*QRCH(I4)*B/A1/A
     21/A1)+(1.0/GRAV*QRCH(I3)*QRCH(I3)*B/A/A/A)-(2.0/3.0*RCHPR(KRCH,3)*
     3ROUGH(KRCH,2)*ROUGH(KRCH,2)*QRCH(I3)*QRCH(I3)*(P**0.33333)/(A**3.3
     43333)*DP)+(1.66667*RCHPR(KRCH,3)*ROUGH(KRCH,2)*ROUGH(KRCH,2)*QRCH(
     5I3)*QRCH(I3)*B*(P**1.33333)/(A**4.33333))
      ODEPTH=DEPTH(KRCH,I3)
      DEPTH(KRCH,I3)=DEPTH(KRCH,I3)-(F/DF)
      IF(DEPTH(KRCH,I3).LT.0.001) GO TO 4007
      DIFF=ABS(ODEPTH-DEPTH(KRCH,I3))
      IF(DIFF.GT.0.00001) GO TO 120
      VEL(KRCH,I3)=QRCH(I3)/AREA(DEPTH(KRCH,I3))
  110 CONTINUE
      DQLAT=QLAT(KRCH)
      CALL HYDROL
      DIFF2=ABS(DQLAT-QLAT(KRCH))
      IF(DIFF2.LT.0.00000001) GO TO 99
      N12=N12+1
      GO TO 444
   99 CONTINUE
      DO 160 L7=1,I
      ADEPT(KRCH,L7)=DEPTH(KRCH,L7)
      AVEL(KRCH,L7)=VEL(KRCH,L7)
  160 CONTINUE
      CALL BWPREP
      I6=KRCH
      IF (IREP(I6,2).EQ.2) GO TO 100
      WRITE(4,3000)I6
 3000 FORMAT (1X,'REACH NUMBER=',I2)
      WRITE(4,3003)
 3003 FORMAT (1X,'LENGTH',4X,'NR. OF NODES',4X,'SECTION LENGTH')
      WRITE(4,3001) RCHPR(I6,1),RCHPR(I6,2),RCHPR(I6,3)
 3001 FORMAT (1X,F8.2,2X,F6.2,9X,F6.2)

      DUM2=RCHPR(I6,2)+0.5
      L20=IFIX(DUM2)
      WRITE(4,3004)
 3004 FORMAT (1X,'NODE',3X,'ELEVATION',4X,'DEPTH',6X,'VELOCITY',6X,'Q')
      DO 140 I=1,L20
      WRITE(4,3002) I,ELEV(I6,I),DEPTH(I6,I),VEL(I6,I),QRCH(I)
 3002 FORMAT (1X,I2,5X,F8.2,5X,F6.4,5X,F6.4,5X,F6.4)
  140 CONTINUE
  100 CONTINUE
      RETURN
 4000 CONTINUE
      PRINT 4001
 4001 FORMAT(/,1X,'INSTABILITY IN BACKWATER CALCULATION.  PROG
     +RAM STOPS.')
      PRINT 4002,I3,KRCH
 4002 FORMAT(1X,'NODE=',I4,'OF REACH=',I4)
      GO TO 4009
 4005 CONTINUE
      PRINT 4006,KRCH
 4006 FORMAT (/,1X,'INSTABILITY OF QLAT CALL IN REACH NO.: ',I3,
     +'  PROGRAM STOPS.')
      GO TO 4009
 4007 CONTINUE
```

```
      PRINT 4008,KRCH
 4008 FORMAT (/,1X,'DEPTH APPROACHING ZERO IN REACH NO.: ',I3,
     +'  PROGRAM STOPS.')
 4009 CONTINUE
      END
C
C
C
C
C
      SUBROUTINE BWPREP
C
C     THIS SUBROUTINE WILL DIRECT THE PROGRAM TO THE APPROPRIATE
C     UBC/DBC SUBROUTINE.  BY SETTING IQ=4, IT WILL ACTIVATE THE
C     UBC...D AND DBC...D (SUB-)SUBROUTINES.
C
      COMMON /CKRCH/KRCH
      COMMON /CIQ/IQ
      IQ=4
      CALL UBC
      CALL DBC
      RETURN
      END

C
C
C
C
C
      SUBROUTINE BCPREP
C
C     THIS SUBROUTINE UPDATES THE DEPTH AND VELOCITY PARAMETERS AT
C     EACH NODE IN MATRIX 'DEPTH' AND 'VEL' UPON THE RESULTING
C     CALCULATED VALUES BY SUBROUTINE 'BACKW'.
C     (SECOND LEVEL SUBROUTINE)
C
      COMMON /CDEPTH/DEPTH(10,21)
      COMMON /CIQ/IQ
      COMMON /CVEL/VEL(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CRCHPR/RCHPR(10,5)
      IQ=2
      CALL UBC
      CALL DBC
      DUM1=RCHPR(KRCH,2)+0.5
      L=IFIX(DUM1)
      DO 550 J13=1,L
      DEPTH(KRCH,J13)=ADEPT(KRCH,J13)
      VEL(KRCH,J13)=AVEL(KRCH,J13)
  550 CONTINUE
      RETURN
      END
C
C
C
C
C
      SUBROUTINE COCALC(NREACH)
C
C     THIS SUBROUTINE PERFORMS THE INITIALIZATION OF THE
C     UBC/DBC PARAMETERS.  (IN PARTICULAR THE DFN/DYN, DFN/DVN AND
C     THE RESIDUAL -RK)
C
      COMMON /CKRCH/KRCH
      COMMON /CIQ/IQ
      IQ=3
      DO 100 K=1,NREACH
      KRCH=K

      CALL UBC
      CALL DBC
  100 CONTINUE
      RETURN
      END
C
C
C
C
C
      SUBROUTINE UBC
C
C     SUBROUTINE UBC WILL DIRECT THE PROGRAM DEPENDING UPON THE
C     TYPE OF DIVERSION STRUCTURE. PRESENTLY ONLY TYPE '1' IS VALID
C     AS AN UPSTREAM BOUNDARY CONDITION TYPE.
C     (THIRD LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CKRCH/KRCH
      NNRCH=KRCH
  570 CONTINUE
      IF(UBCPR(NNRCH,2).GT.1.1) GO TO 580
      CALL UBC01
      GO TO 620
  580 CONTINUE
      IF(UBCPR(NNRCH,2).GT.2.1) GO TO 590
      CALL UBC02
      GO TO 620
  590 CONTINUE
      IF(UBCPR(NNRCH,2).GT.3.1) GO TO 600
      CALL UBC03
      GO TO 620
  600 CONTINUE
      IF(UBCPR(NNRCH,2).GT.4.1) GO TO 610
      CALL UBC04
      GO TO 620
  610 CONTINUE
      PRINT 4300
 4300 FORMAT (/,1X,'UPSTREAM BOUNDARY CONDITION TYPE NO. IS IN ERROR.')
      PRINT 4301,UBCPR(NNRCH,2)
 4301 FORMAT (1X,'PRESENTLY IT IS EQUAL TO',5X,F2.0)
      PRINT 4302
 4302 FORMAT(1X,'IF YOU WISH TO TERMINATE THE SIMULATION INPUT THE')
      PRINT 4303
 4303 FORMAT (1X,'REAL NUMBER,"999.0".')
      PRINT 4304

 4304 FORMAT (1X,'IF YOU WISH TO CONTINUE THE SIMULATION, PLEASE ')
      PRINT 4305
 4305 FORMAT(1X,'ENTER A REPLACEMENT UBC NO.,(REAL TYPE), AT THE
     + PROMPT.')
      READ *,UBCPR(NNRCH,2)
      IF(UBCPR(NNRCH,2).LE.10.) GO TO 570
      GO TO 621
  620 CONTINUE
      RETURN
  621 CONTINUE
      STOP
      END
C
C
C
      SUBROUTINE UBC01
C
C     SUBROUTINE UBC01 IS INITIALIZED BY THE TYPE '1' UPSTREAM
C     BOUNDARY CONDITION AND DIRECT THE PROGRAM UPON THE PRESENT
C     VALUE ASSIGNED TO "IQ". THE VALUE OF IQ IS INITIALLY ASSIGNED
C     TO BE 2 BY SUBROUTINE BCPREP, IQ=3 BY SUBROUTINE HYDRT AND
C     IQ=1 BY SUBROUTINE OPERAT.
C     (FOURTH LEVEL SUBROUTINE)
```

```
C
      COMMON /CIQ/IQ
      IF(IQ.LT.4) GO TO 660
      CALL UBC01D
      GO TO 650
  660 CONTINUE
      IF(IQ.LT.3) GO TO 630
      CALL UBC01A
      GO TO 650
  630 CONTINUE
      IF(IQ.LT.2) GO TO 640
      CALL UBC01B
      GO TO 650
  640 CONTINUE
      CALL UBC01C
  650 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE UBC01A
C
C     THIS SUBROUTINE UPDATES THE UBCPR MATRIX.
C     NEW VALUES ARE ASSIGNED TO PARAMETERS UBCPR X,6,7&8
C     REPRESENTING DGO/DY1, DGO/DY1 AND -RK2,0 RESPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      FND=DAREA(ADEPT(KRCH,1))
      FNA=AREA(ADEPT(KRCH,1))
      UBCPR(KRCH,6)=AVEL(KRCH,1)*FND
      UBCPR(KRCH,7)=FNA
      UBCPR(KRCH,8)=-(AVEL(KRCH,1)*FNA-UBCPR(KRCH,4))
      RETURN
      END
C
C
C
      SUBROUTINE UBC01B
C
C     THIS SUBROUTINE UPDATES THE UBCPR MATRIX.
C     A NEW VALUE IS ASSIGNED TO PARAMETER X,5 REPRESENTING THE
```

```
C     DEPTH Y1.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CAVEL/AVEL(10,21)
      UBCPR(KRCH,4)=AREA(ADEPT(KRCH,1))*AVEL(KRCH,1)
      UBCPR(KRCH,5)=ADEPT(KRCH,1)
      UBCPR(KRCH,10)=UBCPR(KRCH,4)
      RETURN
      END
C
C
C
      SUBROUTINE UBC01C
C
C     THIS SUBROUTINE IS EXECUTED BY SUBROUTINE OPERAT AND
C     UPDATES THE UBCPR MATRIX BY ASSIGNING A NEW INFLOW VALUE TO
C     UBCPR X,4&10
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)

      COMMON /CKRCH/KRCH
      NNRCH=KRCH
      PRINT 4400
 4400 FORMAT (/,1X,'YOU ARE OPERATING THE INLET STRUCTURE TO REACH NUMBE
     +R "1".')
      CALL TIME
      PRINT 4404,UBCPR(NNRCH,4)
 4404 FORMAT(/,1X,'THE PRESENT INFLOW TO THE CANAL IS:  ',F12.5)
      PRINT 4407
 4407 FORMAT(/,1X,'IF YOU WISH TO CHANGE THE INFLOW TO THE CANAL, INPUT
     +1; OTHERWISE, INPUT 999.')
      READ *,TEST
      IF(TEST.GT.998) GO TO 40
      PRINT 4402
 4402 FORMAT (1X,'PLEASE INPUT THE NEW INFLOW TO THE CANAL')
      PRINT 4403
 4403 FORMAT (1X,'WHEN PROMPTED; (AS A REAL VARIABLE).')
      READ *,UBCPR(NNRCH,4)
      UBCPR(NNRCH,10)=UBCPR(NNRCH,4)
   40 CONTINUE
      RETURN
      END
C
```

```
C
C
      SUBROUTINE UBC01D
C
C     THIS SUBROUTINE UPDATES THE UBCPR MATRIX.
C     A NEW VALUE IS ASSIGNED TO UBCPR X,4,5&10 REPRESENTING
C     THE NEW Q, DEPTH Y AND THE DOWNSTREAM DISCHARGE QN.
C     (THIRD LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CAVEL/AVEL(10,21)
      UBCPR(KRCH,4)=AREA(ADEPT(KRCH,1))*AVEL(KRCH,1)
      UBCPR(KRCH,5)=ADEPT(KRCH,1)
      UBCPR(KRCH,10)=UBCPR(KRCH,4)
      RETURN
      END
C
C
C
      SUBROUTINE UBC02
C
C     THIS SUBROUTINE IS INITIALIZED BY THE TYPE "2" UPSTREAM
C     BOUNDARY CONDITION, HENCE INLET TO CANAL-REACHES OTHER
C     THAN #1.  INFLOW IS DETERMINED BY THE OUTFLOW FROM THE
C     UPSTREAM REACH.  IT DIRECTS THE PROGRAM UPON THE PRESENT
C     VALUE OF "IQ", WHICH IS INITIALLY ASSIGNED TO BE 2 BY
C     SUBROUTINE BCPREP, IQ=3 BY SUBROUTINE HYDRT AND IQ=1 BY
C     SUBROUTINE OPERAT. (NOT OPERABLE -¢ RETURNS TO CONTINUE
C     WITH THE PROGRAM.)
C     (FOURTH LEVEL SUBROUTINE)
C
      COMMON /CIQ/IQ
      IF(IQ.LT.4) GO TO 560
      CALL UBC02D
      GO TO 550
  560 CONTINUE
      IF(IQ.LT.3) GO TO 530
      CALL UBC02A
      GO TO 550
  530 IF(IQ.LT.2) GO TO 550
      CALL UBC02B
  550 CONTINUE
      RETURN
      END
```

```
C
C
C
      SUBROUTINE UBC02A
C
C     THIS SUBROUTINE UPDATES THE UBCPR MATRIX.
C     NEW VALUES ARE ASSIGNED TO UBCPR X,6,7&8, REPRESENTING
C     DGO/DY1, DGO/DV1 AND -RK2,0 RESPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      I=KRCH-1
      UBCPR(KRCH,4)=DBCPR(I,20)
      FND=DAREA(ADEPT(KRCH,1))
      FNA=AREA(ADEPT(KRCH,1))
      UBCPR(KRCH,6)=AVEL(KRCH,1)*FND
      UBCPR(KRCH,7)=FNA
      UBCPR(KRCH,8)=-(AVEL(KRCH,1)*FNA-UBCPR(KRCH,4))
      RETURN

      END
C
C
C
      SUBROUTINE UBC02B
C
C     THIS SUBROUTINE UPDATES THE UBC MATRIX.
C     NEW VALUES ARE ASSIGNED TO UBCPR X,4,10&5, REPRESENTING
C     NEW Q, INFLOW FROM UPSTREAM REACH AND THE DEPTH Y
C     RESPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      UBCPR(KRCH,4)=AREA(ADEPT(KRCH,1))*AVEL(KRCH,1)
      UBCPR(KRCH,5)=ADEPT(KRCH,1)
      UBCPR(KRCH,10)=UBCPR(KRCH,4)
      RETURN
      END
C
C
```

```
C
      SUBROUTINE UBC02D
C
C     THIS SUBROUTINE UPDATES THE UBCPR & DBCPR MATRICES.
C     A NEW VALUE IS ASSIGNED TO UBCPR X,4,5&10 REPRESENTING
C     THE NEW Q, DEPTH Y AND THE DOWNSTREAM QN RESPECTIVELY.
C     IT ALSO PERFORMS THE ASSIGNMENT OF THE VALUE OF Q DOWNSTREAM
C     (UBCPR X,10) TO THE UPSTREAM Q (DBCPR X,10)
C     (THIRD LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      UBCPR(KRCH,4)=AREA(ADEPT(KRCH,1))*AVEL(KRCH,1)
      UBCPR(KRCH,5)=ADEPT(KRCH,1)
      UBCPR(KRCH,10)=UBCPR(KRCH,4)
      K13=KRCH-1
      DBCPR(K13,10)=UBCPR(KRCH,10)
      DBCPR(K13,4)=DBCPR(K13,10)+DBCPR(K13,27)
      DBCPR(K13,5)=DBCPR(K13,15)+(DBCPR(K13,10)/DBCPR(K13,16)/DBCPR(K13
     1,11))**0.6666667

      RETURN
      END
C
C
C
      SUBROUTINE UBC03
      RETURN
      END
C
C
C
      SUBROUTINE UBC04
      RETURN
      END
C
C
C
C
C
      SUBROUTINE DBC
C
C     SUBROUTINE DBC WILL DIRECT THE PROGRAM DEPENDING UPON THE TYPE
C     OF STRUCTURE. PRESENTLY ONLY TYPE '1' IS VALID REPRESENTING A
```

```
C     WEIR TYBE DOWNSTREAM BOUNDARY CONDITION.
C     (THIRD LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CKRCH/KRCH
      NNRCH=KRCH
  660 CONTINUE
      IF(DBCPR(NNRCH,2).GT.1.1) GO TO 670
      CALL DBC01
      GO TO 710
  670 CONTINUE
      IF(DBCPR(NNRCH,2).GT.2.1) GO TO 680
      CALL DBC02
      GO TO 710
  680 CONTINUE
      IF(DBCPR(NNRCH,2).GT.3.1) GO TO 690
      CALL DBC03
      GO TO 710
  690 CONTINUE
      IF(DBCPR(NNRCH,2).GT.4.1) GO TO 700
      CALL DBC04
      GO TO 710
  700 CONTINUE

      PRINT 4500
 4500 FORMAT (/,1X,'THE DOWNSTREAM BOUNDARY CONDITION TYPE NUMBER
     + IS IN ERROR.')
      PRINT 4501,DBCPR(NNRCH,2)
 4501 FORMAT (1X,' PRESENTLY, IT IS ',F2.0,'.')
      PRINT 4502
 4502 FORMAT (1X,'YOU MAY INPUT A REPLACEMENT TYPE NO. AT THE ')
      PRINT 4503
 4503 FORMAT (1X,'PROMPT, (AS A REAL NO.), OR YOU MAY TERMINATE')
      PRINT 4504
 4504 FORMAT (1X,'THE SESSION BY INPUTTING THE NO., "999.0".')
      READ *,DBCPR(NNRCH,2)
      IF(DBCPR(NNRCH,2).LT.10.0) GO TO 660
      GO TO 661
  710 CONTINUE
      RETURN
  661 CONTINUE
      STOP
      END
C
C
C
      SUBROUTINE DBC01
```

```
C
C     THIS SUBROUTINE IS INITIALIZED BY THE TYPE '1' DOWNSTREAM
C     BOUNDARY CONDITION AND DIRECT THE PROGRAM UPON THE PRESENT
C     VALUE ASSIGNED TO IQ. THE VALUE OF IQ IS INITIALLY ASSIGNED
C     TO BE 2 BY SUBROUTINE BCPREP, IQ=3 BY SUBROUTINE HYDRT AND
C     IQ=1 BY SUBROUTINE OPERAT.
C     (FOURTH LEVEL SUBROUTINE)
C
      COMMON /CIQ/IQ
      IF(IQ.NE.4) GO TO 750
      CALL DBC01D
      GO TO 740
  750 CONTINUE
      IF(IQ.NE.3) GO TO 720
      CALL DBC01A
      GO TO 740
  720 CONTINUE
      IF(IQ.NE.2) GO TO 730
      CALL DBC01B
      GO TO 740
  730 CONTINUE
      CALL DBC01C
  740 CONTINUE

      RETURN
      END
C
C
C
      SUBROUTINE DBC01A
C
C     THIS SUBROUTINE UPDATES THE DBCPR MATRIX.
C     NEW VALUES ARE ASSIGNED TO PARAMETERS DBCPR X,6,7&8
C     REPRESENTING DFN/DYNJ+1, DFN/DVNJ+1 AND -RK1,N REPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CRCHPR/RCHPR(10,5)
      DUM1=RCHPR(KRCH,2)+0.5
      I6=IFIX(DUM1)
      FNA=AREA(ADEPT(KRCH,I6))
      FND=DAREA(ADEPT(KRCH,I6))
      IF((ADEPT(KRCH,I6)-DBCPR(KRCH,15)).LE.0.0) GO TO 750
      DBCPR(KRCH,6)=1.5*DBCPR(KRCH,16)*DBCPR(KRCH,11)*(ADEPT(KRCH,I6)-DB
     1CPR(KRCH,15))**0.5-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=DBCPR(KRCH,16)*DBCPR(KRCH,11)*(ADEPT(KRCH,I6)-DBCPR(
     1KRCH,15))**1.5-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      GO TO 760
  750 CONTINUE
      DBCPR(KRCH,6)=-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=AVEL(KRCH,I6)*FNA
  760 CONTINUE
      DBCPR(KRCH,7)=-FNA
      RETURN
      END
C
C
C
      SUBROUTINE DBC01B
C
C     THIS SUBROUTINE UPDATES THE DBCPR MATRIX.
C     NEW VALUES ARE ASSIGNED TO PARAMETERS DBCPR X,4,10,18,5&19
C     REPRESENTING: TOTAL QN, ACTUAL QN TO D.S. REACH, OBSERVED QN
C     TO D.S. REACH, DEPTH YN AND MINIMUM ALLOWABLE FLOW TO
C     D.S. REACH RESPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)

C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CRCHPR/RCHPR(10,5)
      DUM1=RCHPR(KRCH,2)+0.5
      I6=IFIX(DUM1)
      FNA=AREA(ADEPT(KRCH,I6))
      CALL RANDOM(RND)
      DRND=RND
      DBCPR(KRCH,4)=AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,10)=DBCPR(KRCH,4)
      DBCPR(KRCH,18)=DBCPR(KRCH,10)*(1.0+DBCPR(KRCH,17)*(DRND-0.5))
      DBCPR(KRCH,5)=ADEPT(KRCH,I6)
      IF(DBCPR(KRCH,19).GE.DBCPR(KRCH,10)) GO TO 770
      DBCPR(KRCH,20)=DBCPR(KRCH,10)
      GO TO 780
  770 CONTINUE
      DBCPR(KRCH,20)=DBCPR(KRCH,19)
  780 CONTINUE
      RETURN
      END
```

```
C
C
C
      SUBROUTINE DBC01C
C
C     THIS SUBROUTINE WILL NOTIFY THE CONDITION AT THE DOWNSTREAM
C     STRUCTURE.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CKRCH/KRCH
      NNRCH=KRCH
      PRINT 4604,NNRCH
 4604 FORMAT(/,1X,'DOWNSTREAM STRUCTURE OF REACH NO.',1X,I4)
      CALL TIME
      PRINT 4600
 4600 FORMAT (/,1X,'THERE ARE NO OPERABLE FEATURES ON THIS DROP STRUCTUR
     1E.')
      PRINT 4601
 4601 FORMAT (1X,'HOWEVER, YOUR ESTIMATE OF THE FLOW PASSING THIS ')
      PRINT 4602,DBCPR(NNRCH,18)
 4602 FORMAT (1X,'DROP STRUCTURE IS ',1X,F6.3)
      PRINT 4603,DBCPR(NNRCH,5)

 4603 FORMAT (1X,'THE DEPTH OF THE WATER UPSTREAM OF THE DROP IS ',2X,F8
     1.4,'.')
      RETURN
      END
C
C
C
      SUBROUTINE DBC01D
C
C     THIS SUBROUTINE UPDATES THE DBCPR MATRIX.
C     A NEW VALUE IS ASSIGNED TO THE DBCPR X,4,5,18&20 REPRESENTING
C     QN, YN, OBSERVED Q TO D.S. REACH AND THE NUMERICAL FLOW TO
C     THE D.S. REACH.   (THIRD LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CKRCH/KRCH
      DBCPR(KRCH,4)=DBCPR(KRCH,10)
      DBCPR(KRCH,18)=DBCPR(KRCH,10)
      DBCPR(KRCH,20)=DBCPR(KRCH,10)
      DBCPR(KRCH,5)=DBCPR(KRCH,15)+(DBCPR(KRCH,10)/DBCPR(KRCH,16)/DBCPR(
     1KRCH,11))**0.6666667
      RETURN
      END
```

```
C
C
C
      SUBROUTINE DBC02
C
C     THIS SUBROUTINE IS INITIALIZED BY THE TYPE "2" DOWNSTREAM
C     BOUNDARY CONDITION (ADJUSTABLE WEIR-CHECK AND ONE FARM TURNOUT)
C     IT DIRECTS THE PROGRAM UPON THE PRESENT VALUE OF 'IQ'.  THE
C     VALUE OF IQ IS INITIALLY ASSIGNED TO BE 2 BY SUBROUTINE BCPREP
C     IQ=3 BY SUBROUTINE HYDRT AND IQ=1 BY SUBROUTINE OPERAT.
C     (FOURTH LEVEL SUBROUTINE)
C
      COMMON /CIQ/IQ
      IF(IQ.NE.4) GO TO 850
      CALL DBC02D
      GO TO 840
  850 CONTINUE
      IF(IQ.NE.3) GO TO 820
      CALL DBC02A
      GO TO 840
  820 CONTINUE
      IF(IQ.NE.2) GO TO 830
      CALL DBC02B

      GO TO 840
  830 CONTINUE
      CALL DBC02C
  840 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE DBC02A
C
C     THIS SUBROUTINE UPDATES THE DBC MATRIX.
C     NEW VALUES ARE ASSIGNED TO PARAMETERS DBCPR X,6,7&8
C     REPRESENTING DFN/DYNJ+1, DFN/DVNJ+1 AND -RK1,N RESPECTIVELY.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CRCHPR/RCHPR(10,5)
      DUMM1=RCHPR(KRCH,2)+0.5
      I6=IFIX(DUMM1)
      FNA=AREA(ADEPT(KRCH,I6))
      FND=DAREA(ADEPT(KRCH,I6))
      IF((ADEPT(KRCH,I6)-DBCPR(KRCH,15)).LE.0.0) GO TO 60
      IF((ADEPT(KRCH,I6)-(DBCPR(KRCH,21)+DBCPR(KRCH,24))).LE.0.0)
     1GO TO 61
C
C
C     CONDITION #1
C
      DTO1=ADEPT(KRCH,I6)-DBCPR(KRCH,15)
      DTO2=ADEPT(KRCH,I6)-DBCPR(KRCH,21)
      DTO3=DTO2-DBCPR(KRCH,24)
      DBCPR(KRCH,6)=1.5*DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**0.5)+
     14.4294469*DBCPR(KRCH,25)*DBCPR(KRCH,22)*((DTO2**.5)-(DTO3**.5))
     2-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**1.5)+
     12.9529646*DBCPR(KRCH,25)*DBCPR(KRCH,22)*((DTO2**1.5)-
     2(DTO3**1.5))-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
      GO TO 69
   61 CONTINUE
      IF((ADEPT(KRCH,I6)-DBCPR(KRCH,21)).LE.0.0) GO TO 62
C
C
C     CONDITION #2
C
      DTO1=ADEPT(KRCH,I6)-DBCPR(KRCH,15)
      DTO2=ADEPT(KRCH,I6)-DBCPR(KRCH,21)
      DBCPR(KRCH,6)=1.5*DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**0.5)
     1+4.4294469*DBCPR(KRCH,25)*DBCPR(KRCH,22)*(DTO2**0.5)
     2-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**1.5)
     1+DBCPR(KRCH,25)*DBCPR(KRCH,22)*(DTO2**1.5)-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
      GO TO 69
   62 CONTINUE
C
C
C     CONDITION #3
C
      DTO1=ADEPT(KRCH,I6)-DBCPR(KRCH,15)
      DBCPR(KRCH,6)=1.5*DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**0.5)
     1-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=DBCPR(KRCH,16)*DBCPR(KRCH,11)*(DTO1**1.5)
     1-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
      GO TO 69
   60 CONTINUE
      IF((ADEPT(KRCH,I6)-(DBCPR(KRCH,24)+DBCPR(KRCH,21))).LE.0.0)
     1GO TO 63
C
C
C     CONDITION #4
C
      DTO2=ADEPT(KRCH,I6)-DBCPR(KRCH,21)
      DTO3=DTO2-DBCPR(KRCH,24)
      DBCPR(KRCH,6)=4.4294469*DBCPR(KRCH,25)*DBCPR(KRCH,22)*
     1((DTO2**0.5)-(DTO3**0.5))-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=2.9529646*DBCPR(KRCH,25)*DBCPR(KRCH,22)*
     1((DTO2**1.5)-(DTO3**1.5))-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
      GO TO 69
   63 CONTINUE
      IF((ADEPT(KRCH,I6)-DBCPR(KRCH,21)).LE.0.0) GO TO 64
C

C
C     CONDITION #5
C
      DTO2=ADEPT(KRCH,I6)-DBCPR(KRCH,21)
      DBCPR(KRCH,6)=4.4294469*DBCPR(KRCH,25)*DBCPR(KRCH,22)*
     1(DTO2**0.5)-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=2.9529646*DBCPR(KRCH,25)*DBCPR(KRCH,22)*
     1(DTO2**1.5)-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
      GO TO 69
   64 CONTINUE
C
C
C     CONDITION #6
C
      DBCPR(KRCH,6)=-AVEL(KRCH,I6)*FND
      DBCPR(KRCH,8)=-AVEL(KRCH,I6)*FNA
      DBCPR(KRCH,8)=-DBCPR(KRCH,8)
      DBCPR(KRCH,7)=-FNA
   69 CONTINUE
      RETURN
      END
```

```
C
C
C
      SUBROUTINE DBC02B
C
C     THIS SUBROUTINE UPDATES THE DBCPR MATRIX.
C     NEW VALUES ARE ASSIGNED TO PARAMETERS DBCPR 4,27,9,10,18,5,19,20,
C     31,33 AND 37 REPRESENTING DISCHARGE, ACTUAL FARM DELIVERY, USEFUL
C     FARM DELIVERY, ACTUAL FLOW TO D.S. REACH, DEPTH OF FLOW AT END OF
C     REACH, MINIMUM FLOW TO DOWNSTREAM REACH, NUMERICAL FLOW TO
C     DOWNSTREAM REACH, OBSERVED DEPTH UPSTREAM OF CHECK, OBSERVED
C     FARM WATER DELIVERY AND TOTAL USEFUL FLOW TO DOWNSTREAM
C     REACH.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CNREACH/NREACH
      COMMON /CQLAT/QLAT(10)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CRCHPR/RCHPR(10,5)
      DUM1=RCHPR(KRCH,2)+0.5

      I6=IFIX(DUM1)
      FNA=AREA(ADEPT(KRCH,I6))
C
C     UPDATE THE TOTAL FLOW AT THE END OF THE REACH, PARAMETER 4.
C
      DBCPR(KRCH,4)=AVEL(KRCH,I6)*FNA
C
C     UPDATE THE ACTUAL FLOW THROUGH THE FARM DELIVERY, PARAMETER 27.
C
      DTO2=ADEPT(KRCH,I6)-DBCPR(KRCH,21)
      DTO3=DTO2-DBCPR(KRCH,24)
      IF(DTO2.LE.0.0) GO TO 951
      IF(DTO3.LE.0.0) GO TO 952
C
C     CONDITIONS 1 AND 4 -- ORIFICE FLOW
C
      DBCPR(KRCH,27)=2.9529646*DBCPR(KRCH,25)*DBCPR(KRCH,22)*(DTO2**1.5-
     1DTO3**1.5)
      GO TO 953
  952 CONTINUE
C
C     CONDITIONS 2 AND 5  -- WIER FLOW
C
```

```
      DBCPR(KRCH,27)=2.9529646*DBCPR(KRCH,25)*DBCPR(KRCH,22)*(DTO2**1.5)
      GO TO 953
  951 CONTINUE
C
C     CONDITIONS 3 AND 6  --  NO FLOW
C
      DBCPR(KRCH,27)=0.0
  953 CONTINUE
C
C
C     DETERMINE UTILITY OF ACTUAL FARM WATER DELIVERY
C     UPDATE PARAMETER 9.
C
      TEST1=DBCPR(KRCH,28)*DBCPR(KRCH,26)
      IF(DBCPR(KRCH,27).GE.TEST1) GO TO 900
      DBCPR(KRCH,9)=DBCPR(KRCH,27)
      TEST2=DBCPR(KRCH,29)*DBCPR(KRCH,26)
      IF(DBCPR(KRCH,27).LT.TEST2) GO TO 901
      GO TO 902
  901 CONTINUE
      DBCPR(KRCH,9)=0.00000
      GO TO 902
  900 CONTINUE

      DBCPR(KRCH,9)=DBCPR(KRCH,28)*DBCPR(KRCH,26)
  902 CONTINUE
C
C     CALCULATE THE ACTUAL FLOW TO THE D.S. REACH, PARAMETER 10.
C
      DBCPR(KRCH,10)=DBCPR(KRCH,4)-DBCPR(KRCH,27)
C
C     CALCULATE THE OBSERVED FLOW TO THE D.S. REACH, PARAMETER 18.
C
      DBCPR(KRCH,18)=DBCPR(KRCH,10)*(1+DBCPR(KRCH,17))
C
C     UPDATE THE ACTUAL DEPTH OF WATER AT THE END OF THE REACH, PARAMETER 5.
C
      DBCPR(KRCH,5)=ADEPT(KRCH,I6)
C
C     DETERMINATION OF NUMERICAL FLOW TO DOWNSTREAM REACH
C
C     FIRST, DETERMINE THE MAGNITUDE OF THE MIN. FLOW TO D.S. REACH.
C     UPDATE PARAMETER 19.
C
      IF(DBCPR(KRCH,39).GT.1.0) GO TO 1000
      IF((KRCH-NREACH).LE.0) GO TO 1000
      K=KRCH+1
      QD1=QLAT(K)*RCHPR(K,1)
      QDT=DBCPR(KRCH,40)-QD1
      IF(QDT.GE.0.0) GO TO 1000
      DBCPR(KRCH,19)=QD1
      GO TO 1001
 1000 CONTINUE
      DBCPR(KRCH,19)=DBCPR(KRCH,40)
 1001 CONTINUE
C
C     SECOND, SET NUMERICAL FLOW TO D.S. REACH.
C     UPDATE PARAMETER 20.
C
      IF(DBCPR(KRCH,19).GE.DBCPR(KRCH,10)) GO TO 870
      DBCPR(KRCH,20)=DBCPR(KRCH,10)
      GO TO 880
  870 CONTINUE
      DBCPR(KRCH,20)=DBCPR(KRCH,19)
  880 CONTINUE
C
C     CALCULATE THE OBSERVED DEPTH, PARAMETER 31.
C
      DBCPR(KRCH,31)=DBCPR(KRCH,5)*(1+DBCPR(KRCH,30))
C
C     CALCULATE THE OBSERVED FARM WATER DELIVERY, PARAMETER 33.
C
      DBCPR(KRCH,33)=DBCPR(KRCH,27)*(1+DBCPR(KRCH,32))
C
C     DETERMINE THE USEFUL FLOW TO DOWNSTREAM REACH, PARAMETER 37.
C
      TEST3=DBCPR(KRCH,34)*DBCPR(KRCH,35)
      IF(DBCPR(KRCH,20).GE.TEST3) GO TO 905
      DBCPR(KRCH,37)=DBCPR(KRCH,20)
      TEST4=DBCPR(KRCH,34)*DBCPR(KRCH,36)
      IF(DBCPR(KRCH,20).LT.TEST4) GO TO 906
      GO TO 907
  905 CONTINUE
      DBCPR(KRCH,37)=DBCPR(KRCH,34)*DBCPR(KRCH,35)
      GO TO 907
  906 CONTINUE
      DBCPR(KRCH,37)=0.00000
  907 CONTINUE
      RETURN
      END
C
C
C
```

```
      SUBROUTINE DBC02C
C
C     THIS SUBROUTINE ALLOWS THE USER TO OPERATE THE DBC-TYPE "2"
C     STRUCTURE, CONSISTING OUT OF AN ADJUSTABLE WEIR-CHECK AND
C     AN ADJUSTABLE FARM TURNOUT.
C     (FIFTH LEVEL SUBROUTINE)
C
      COMMON /CKRCH/KRCH
      COMMON /CNDAY/NDAY
      COMMON /CATDAY/ATDAY
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CRCHPR/RCHPR(10,5)
      NNRCH=KRCH
      PRINT 4700,NNRCH
 4700 FORMAT (/,1X,'YOU ARE PRESENTLY AT REACH NO.: ',1X,I2)
      CALL TIME
      PRINT 4702
 4702 FORMAT (1X,'DBC-TYPE IS: "2" (WEIR-CHECK / TURNOUT)')
      PRINT 4703,DBCPR(NNRCH,31)
 4703 FORMAT (1X,'OBSERVED DEPTH OF WATER = ',F6.3)
      PRINT 4704,DBCPR(NNRCH,10)
 4704 FORMAT (1X,'OBSERVED FLOW TO D.S. REACH = ',F6.3)
      PRINT 4705,DBCPR(NNRCH,26)

 4705 FORMAT (1X,'PRESENT FARM REQUIREMENT = ',F6.3)
      PRINT 4706,DBCPR(NNRCH,33)
 4706 FORMAT (1X,'OBSERVED FARM DELIVERY = ',F6.3)
      PRINT 4707,DBCPR(NNRCH,9)
 4707 FORMAT (1X,'USEFUL FARM DELIVERY = ',F6.3)
      PRINT 4708
 4708 FORMAT (/,1X,'IF YOU WISH TO CHANGE FARM REQUIREMENTS INPUT 1 ; OT
     +HERWISE INPUT 999')
      READ *,I1
      I1=I1
      IF(I1.GT.998) GO TO 891
      PRINT 4709
 4709 FORMAT (/,1X,'INPUT THE NEW FARM REQUIREMENT.')
      READ *,DBCPR(NNRCH,26)
      DBCPR(NNRCH,26)=DBCPR(NNRCH,26)
  891 CONTINUE
      PRINT 4710
 4710 FORMAT (/,1X,'IF YOU WISH TO OPERATE THE WEIR-CHECK, INPUT 1 ; OTH
     +ERWISE INPUT 999')
      READ *,I2
      I2=I2
      IF(I2.GT.998) GO TO 892
      PRINT 4711
 4711 FORMAT (/,1X,'OPERATION OF WEIR-CHECK IS NOW PERMITTED.')
      PRINT 4712,DBCPR(NNRCH,13)
 4712 FORMAT (1X,'THE MAXIMUM HEIGHT IS: ',F6.3)
      PRINT 4713,DBCPR(NNRCH,12)
 4713 FORMAT (1X,'THE MINIMUM HEIGHT IS: ',F6.3)
      PRINT 4714,DBCPR(NNRCH,14)
 4714 FORMAT (1X,'THE HEIGHT INCREMENT IS: ',F6.3)
      PRINT 4715,DBCPR(NNRCH,15)
 4715 FORMAT (1X,'THE CURRENT HEIGHT IS: ',F6.3)
      PRINT 4716
 4716 FORMAT (/,1X,'PLEASE INPUT THE NEW CHECK HEIGHT.')
      READ *,DBCPR(NNRCH,15)
      DBCPR(NNRCH,15)=DBCPR(NNRCH,15)
      IF(DBCPR(NNRCH,15).LE.DBCPR(NNRCH,12)) GO TO 200
      P3=DBCPR(NNRCH,15)
      DD=(P3-DBCPR(NNRCH,12))/DBCPR(NNRCH,14)-0.1
      P4=INT(DD)
      DBCPR(NNRCH,15)=(P4+1)*DBCPR(NNRCH,14)+DBCPR(NNRCH,12)
      GO TO 201
  200 CONTINUE
      DBCPR(NNRCH,15)=DBCPR(NNRCH,12)
  201 CONTINUE
      PRINT 4717,DBCPR(NNRCH,15)

 4717 FORMAT (/,1X,'THE NEW WEIR-CHECK HEIGHT IS: ',F6.3)
  892 CONTINUE
      PRINT 4718
 4718 FORMAT (/,1X,'IF YOU WISH TO OPERATE THE TURNOUT INPUT 1; OTHERWIS
     +E INPUT 999')
      READ *,I5
      I5=I5
      IF(I5.GT.998) GO TO 893
      PRINT 4719
 4719 FORMAT (/,1X,'OPERATION OF THE TURNOUT IS NOW PERMITTED.')
      PRINT 4705,DBCPR(NNRCH,26)
      PRINT 4706,DBCPR(NNRCH,33)
      PRINT 4725,DBCPR(NNRCH,24)
      PRINT 4720,DBCPR(NNRCH,23)
 4720 FORMAT (1X,'MAXIMUM TURNOUT OPENING IS: ',F6.3)
      PRINT 4721
 4721 FORMAT (/,1X,'INPUT THE NEW SELECTED FARM DELIVERY.')
      PRINT 4726
 4726 FORMAT (1X,'THIS IS USUALLY EQUAL TO THE FARM-WATER REQUIREMENTS.'
     +)
      READ *,DBCPR(NNRCH,27)
      DBCPR(NNRCH,27)=DBCPR(NNRCH,27)
      IF((DBCPR(NNRCH,4)-DBCPR(NNRCH,27)).LE.0.0) GO TO 100
      DUM1=((DBCPR(NNRCH,4)-DBCPR(NNRCH,27))/DBCPR(NNRCH,16)/DBCPR(NNRCH
     1,11))**0.6666667+DBCPR(NNRCH,15)
      DTO2=DUM1-DBCPR(NNRCH,21)
      DUMM1=DBCPR(NNRCH,27)/(DBCPR(NNRCH,25)*DBCPR(NNRCH,22)*2.9529646)-
     1DTO2**1.5
C     THE ABOVE CALCULATION IS INTRODUCED TO DETERMINE THE TURNOUT OPENING
C     AT STEADY STATE, HENCE, IT CORRELATES WITH THE NEW SELECTED FARM
C     DELIVERY.  THIS DBC-PARAMETER ¼DBCPR(NNRCH,24)  IS FURTHER NOT BEING
C     UPDATED BY ANY OTHER SUBROUTINE.
      IF (DUMM1.LT.0.0) GO TO 900
      DUMM1=DUMM1**0.6666667
      DBCPR(NNRCH,24)=DTO2+DUMM1
      GO TO 901
  900 CONTINUE
      DUMM1=(ABS(DUMM1))**0.6666667
      DBCPR(NNRCH,24)=DTO2-DUMM1
  901 CONTINUE
      PRINT 4725,DBCPR(NNRCH,24)
 4725 FORMAT (/,1X,'TURNOUT OPENING= ',F8.6)
      IF(DBCPR(NNRCH,24).GT.DBCPR(NNRCH,23)) GO TO 894
      GO TO 893
  894 CONTINUE
      PRINT 4722,DBCPR(NNRCH,27)

 4722 FORMAT (/,1X,'TO DELIVER:',F8.4,1X,'CMS. AT THE TURNOUT REQUIRES
     + AN TURNOUT')
      PRINT 4723,DBCPR(NNRCH,24)
 4723 FORMAT (1X,'OPENING OF: ',F6.3,1X,'M.')
  100 CONTINUE
      PRINT 4724,DBCPR(NNRCH,23)
 4724 FORMAT (1X,'TURNOUT WILL BE SET AT MAXIMUM OPENING OF:',F6.3,
     +1X,'M.')
      DBCPR(NNRCH,24)=DBCPR(NNRCH,23)
  893 CONTINUE
      PRINT 5000
 5000 FORMAT(/,1X,'IF YOU WISH TO CHANGE THE D.S. REQUIREMENTS,')
      PRINT 4999
 4999 FORMAT(/,1X,'INPUT THE NO. 1; OTHERWISE INPUT THE NO. 999.')
      READ *,I6
      IF(I6.GT.998) GO TO 5002
      PRINT 5003,DBCPR(NNRCH,34)
 5003 FORMAT(/,1X,'THE PRESENT D.S. REQUIREMENTS EQUAL:',5X,F8.4)
      PRINT 5004
 5004 FORMAT(/,1X,'INPUT THE NEW D.S. REQUIREMENTS WHEN PROMPTED.')
      READ *,P6
      DBCPR(NNRCH,34)=P6
      PRINT 5005
```

```
5005 FORMAT(/,1X,'THE TOL. LIMITS ON THE D.S. REQUIREMENTS ARE:')
     PRINT 5006,DBCPR(NNRCH,35)
5006 FORMAT(/,1X,'UPPER:',F8.4)
     PRINT 5007,DBCPR(NNRCH,36)
5007 FORMAT(/,1X,'LOWER:',F8.4)
     PRINT 5008
5008 FORMAT(/,1X,'IF YOU TO CHANGE THE TOLERANCE LIMITS, INPUT 1;')
     PRINT 5009
5009 FORMAT(/,1X,'OTHERWISE, INPUT 999.')
     READ *,I7
     IF(I7.GT.998) GO TO 5010
     PRINT 5011
5011 FORMAT(/,1X,'INPUT THE UPPER TOL. LIMIT WHEN PROMPTED.')
     READ *,P7
     DBCPR(NNRCH,35)=P7
     PRINT 5012
5012 FORMAT(/,1X,'INPUT THE LOWER TOL. LIMIT WHEN PROMPTED.')
     READ *,P8
     DBCPR(NNRCH,36)=P8
5010 CONTINUE
5002 CONTINUE
     RETURN
     END
C
C
C
     SUBROUTINE DBC02D
C
C
C    THIS SUBROUTINE UPDATES THE DBCPR MATRIX FOR THE REACH
C    THAT HAS JUST HAD ITS BACKWATER CURVE CALCULATED.
C
C    CORRECT VALUES ARE ASSIGNED TO THE DBCPR PARAMETERS; 4,18,
C    20,31,33,5,24,27,9, AND 37, WHICH CORRESPOND TO THE
C    ACTUAL DEPTH AT THE END OF THE REACH, THE OBSERVED DEPTH AT
C    THE CHECK, THE NUMERICAL FLOW PASSING THE CHECK, THE
C    OBSERVED DEPTH AT THE END OF THE REACH, THE OBSERVED FARM
C    DELIVERY, THE ACTUAL FLOW AT THE END OF THE REACH,
C    THE ACTUAL FARM TURNOUT OPENING, THE ACTUAL FARM WATER
C    DELIVERY, THE TOTAL USEFUL FARM WATER DELIVERY AND THE
C    TOTAL USEFUL FLOW PASSING THE CHECK TO THE DOWNSTREAM REACH.
C
C
     COMMON /CDBCPR/DBCPR(10,60)
     COMMON /CKRCH/KRCH
C
```

```
C     UPDATE THE TOTAL FLOW AT THE END OF THE REACH.
C
      DBCPR(KRCH,4)=DBCPR(KRCH,10)+DBCPR(KRCH,27)
C
C     UPDATE THE OBSERVED FLOW PASSING THE CHECK.
C
      DBCPR(KRCH,18)=DBCPR(KRCH,10)*(1+DBCPR(KRCH,17))
C
C     UPDATE THE NUMERICAL FLOW PASSING THE CHECK TO D.S. REACH.
C
      DBCPR(KRCH,20)=DBCPR(KRCH,10)
C
C     UPDATE THE ACTUAL DEPTH UPSTREAM OF THE CHECK AND T.O.
C
      DBCPR(KRCH,5)=DBCPR(KRCH,15)+(DBCPR(KRCH,10)/DBCPR(KRCH,16)/DBCPR(
     1KRCH,11))**0.6666667
C
C     UPDATE THE OBSERVED DEPTH UPSTREAM OF THE CHECK AND T.O.
C
      DBCPR(KRCH,31)=DBCPR(KRCH,5)*(1+DBCPR(KRCH,30))
C
C     DETERMINE THE FLOW THROUGH THE TURNOUT.
C     IF POSSIBLE THE SPECIFIED FLOW IS PASSED; OTHERWISE,

C     THE MAXIMUM POSSIBLE FLOW IS PASSED.
C     THE TURNOUT OPENING IS SET ACCORDINGLY.
C
C
C     TEST TO SEE IF FLOW THROUGH T.O. IS POSSIBLE.
C
      IF((DBCPR(KRCH,5)-DBCPR(KRCH,21)).GT.0.0) GO TO 500
C
C     NO FLOW THROUGH THE T.O. IS POSSIBLE.
C
      DBCPR(KRCH,27)=0.0
      DBCPR(KRCH,24)=0.0
      GO TO 999
  500 CONTINUE
C
C     FLOW THROUGH THE T.O. IS POSSIBLE.
C
C
C     DETERMINE THE MAXIMUM FLOW POSSIBLE.
C     AT THE CURRENT WATER DEPTH.  THIS WILL OCCUR AT THE
C     MAXIMUM T.O. OPENING POSSIBLE WHILE STILL MAINTAINING
C     ORIFICE FLOW.
C
```

```
      HSTAR=DBCPR(KRCH,5)-DBCPR(KRCH,23)-DBCPR(KRCH,21)
      IF(HSTAR.LE.0.0) GO TO 501
C
C     MAXIMUM T.O. OPENING EQUALS THE MAXIMUM OPENING ALLOWED.
C
      WOMAX=DBCPR(KRCH,23)
      QMAX=DBCPR(KRCH,25)*DBCPR(KRCH,22)*2.9529646*(((DBCPR(KRCH,5)-
     1DBCPR(KRCH,21))**1.5)-((DBCPR(KRCH,5)-DBCPR(KRCH,21)-
     2DBCPR(KRCH,24))**1.5))
      GO TO 502
  501 CONTINUE
C
C     THE MAXIMUM T.O. OPENING IS LESS THAN THE MAXIMUM OPENING ALLOWED.
C
      WOMAX=DBCPR(KRCH,5)-DBCPR(KRCH,21)
      QMAX=DBCPR(KRCH,25)*DBCPR(KRCH,22)*2.9529646*((DBCPR(KRCH,5)-
     1DBCPR(KRCH,21))**1.5)
  502 CONTINUE
C
C     DETERMINE THE ACTUAL FLOW THROUGH THE T.O.
C
      IF((QMAX-DBCPR(KRCH,27)).GT.0.0) GO TO 503
C
C     SELECTED FLOW THROUGH THE T.O. IS NOT POSSIBLE.
C     SET ACTUAL FLOW EQUAL TO THE MAXIMUM POSSIBLE.
C     SET ACTUAL T.O. OPENING TO THE MAXIMUM POSSIBLE.
C
      DBCPR(KRCH,27)=QMAX
      DBCPR(KRCH,24)=WOMAX
  503 CONTINUE
C
C     SELECTED FLOW THROUGH T.O. IS POSSIBLE.
C     CALCULATE THE ACTUAL T.O. OPENING.
C
      DBCPR(KRCH,24)=DBCPR(KRCH,5)-DBCPR(KRCH,21)-((((DBCPR(KRCH,5)-
     1DBCPR(KRCH,21))**1.5)-(DBCPR(KRCH,27)/DBCPR(KRCH,25)/
     2DBCPR(KRCH,22)/2.9529646))**.666667)
  999 CONTINUE
C
C     UPDATE THE OBSERVED FLOW THROUGH THE T.O.
C
      DBCPR(KRCH,33)=DBCPR(KRCH,27)*(1+DBCPR(KRCH,32))
C
C     DETERMINE THE USEFUL FARM WATER DELIVERY.
C
      TEST1=DBCPR(KRCH,28)*DBCPR(KRCH,26)
      IF(DBCPR(KRCH,27).GE.TEST1) GO TO 200
      DBCPR(KRCH,9)=DBCPR(KRCH,27)
      TEST2=DBCPR(KRCH,29)*DBCPR(KRCH,26)
      IF(DBCPR(KRCH,27).LT.TEST2) GO TO 201
      GO TO 202
  201 CONTINUE
      DBCPR(KRCH,9)=0.0000
      GO TO 202
  200 CONTINUE
      DBCPR(KRCH,9)=DBCPR(KRCH,28)*DBCPR(KRCH,26)
  202 CONTINUE
C
C     DETERMINE THE USEFUL FLOW TO THE DOWNSTREAM REACH.
C
      TEST3=DBCPR(KRCH,34)*DBCPR(KRCH,35)
      IF(DBCPR(KRCH,20).GE.TEST3) GO TO 205
      DBCPR(KRCH,37)=DBCPR(KRCH,20)
      TEST4=DBCPR(KRCH,34)*DBCPR(KRCH,36)
      IF(DBCPR(KRCH,20).LT.TEST4) GO TO 206
      GO TO 207
  205 CONTINUE
      DBCPR(KRCH,37)=DBCPR(KRCH,34)*DBCPR(KRCH,35)
      GO TO 207

  206 CONTINUE
      DBCPR(KRCH,37)=0.0000
  207 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE DBC03
      RETURN
      END
C
C
C
      SUBROUTINE DBC04
      RETURN
      END
C
C
C
C
C
      SUBROUTINE RANDOM(RND)
```

```
C
C     THIS SUBROUTINE WILL GENERATE A RANDOM NUMBER, WHICH IS USED
C     TO CALCULATE DBCPR X,18 (-THE OBSERVED QN TO D.S. REACH-)
C
      INTEGER A,X
      DATA I/1/
      IF (I.EQ.0) GO TO 1
      I=0
      M=2**20
      FM=M
      X=566387
      A=2**10+3
    1 X=MOD(A*X,M)
      FX=X
      RND=FX/FM
      RETURN
      END
C
C
C
C
C
      SUBROUTINE REPORT(THYINS)

C
C     THIS SUBROUTINE CONTROLS THE OUTPUT OR OMISSION OF FILES
C     CONTAINING PROCESSING RESULTS. FILE CREATION IS CONTROLLED
C     BY REPORT INDICES, WHICH ARE USER SPECIFIED IN THE 'RCHDAT'
C     PROGRAM.
C     INDEX=1  -¢ OUTPUT IS DESIRED
C     INDEX=2  -¢ OUTPUT IS OMITTED
C     PRESENT CONTROL PARAMETERS FOR REPORT ARE:
C     IREP (KRCH,1) -¢ INITIAL DATA
C     IREP (KRCH,2) -¢ BACKWATER CURVE
C     IREP (KRCH,3) -¢ WEIR CHECK STATUS
C     IREP (KRCH,4) -¢ FARM DELIVERY #1 STATUS
C     IREP (KRCH,5) -¢ FARM DELIVERY #2 STATUS (INACTIVE)
C     IREP (KRCH,6) -¢ NODES STATUS
C     IREP (KRCH,7) -¢ CHANNEL STATUS
C     IREP (KRCH,8) -¢ TIME STATUS
C     IREP (KRCH,9) -¢ WATER BALANCE STATUS
C     IREP (KRCH,10) -¢ FUTURE IMPLEMENTATION
C     IREP (KRCH,11) -¢ DBC PARAMETERS
C     IREP (KRCH,12) -¢ UBC PARAMETERS
C
      DIMENSION WATBAL(20),CHSTAT(4),FD1(8),FD2(8),CHK(5)
      COMMON /CIREP/IREP(10,12)
      COMMON /CKRCH/KRCH
      COMMON /CDEPTH/DEPTH(10,21)
      COMMON /CVEL/VEL(10,21)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CQSEEP/QSEEP(10,20)
      COMMON /CQEVAP/QEVAP(10,20)
      COMMON /CQPREC/QPREC(10,20)
      COMMON /CQODST/QODST(10,20)
      COMMON /CTTHR/TTHR
      COMMON /CRCHPR/RCHPR(10,5)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CCMBAL/CMBAL(10,20)
      IF (IREP(KRCH,9).EQ.2) GO TO 810
      DUM1=RCHPR(KRCH,2)-0.5
      N1=IFIX(DUM1)
      STOR=0.0
      DO 820 N2=1,N1
      STOR=STOR+(AREA(DEPTH(KRCH,N2))+AREA(DEPTH(KRCH,N2+1)))*RCHPR(KRCH
     1,3)/2.0
  820 CONTINUE
      WATBAL(1)=UBCPR(KRCH,4)

      WATBAL(2)=DBCPR(KRCH,4)
      WATBAL(3)=DBCPR(KRCH,10)
      WATBAL(4)=DBCPR(KRCH,20)
      WATBAL(5)=DBCPR(KRCH,9)
      WATBAL(6)=STOR/1000.
      CMBAL(KRCH,6)=STOR
      WATBAL(7)=QSEEP(KRCH,3)*RCHPR(KRCH,1)
      WATBAL(8)=QEVAP(KRCH,3)*RCHPR(KRCH,1)
      WATBAL(9)=QPREC(KRCH,3)*RCHPR(KRCH,1)
      WATBAL(10)=QODST(KRCH,3)*RCHPR(KRCH,1)
      WATBAL(11)=DBCPR(KRCH,19)
      WATBAL(12)=QSEEP(KRCH,3)
      WATBAL(13)=QEVAP(KRCH,3)
      WATBAL(14)=QPREC(KRCH,3)
      WATBAL(15)=QODST(KRCH,3)
      WATBAL(16)=DBCPR(KRCH,37)
      IF(TTHR.LE.0.001) GO TO 600
      CMBAL(KRCH,1)=CMBAL(KRCH,1)+WATBAL(1)*THYINS
      CMBAL(KRCH,2)=CMBAL(KRCH,2)+WATBAL(2)*THYINS
      CMBAL(KRCH,3)=CMBAL(KRCH,3)+WATBAL(3)*THYINS
      CMBAL(KRCH,4)=CMBAL(KRCH,4)+WATBAL(4)*THYINS
      CMBAL(KRCH,5)=CMBAL(KRCH,5)+WATBAL(5)*THYINS
      CMBAL(KRCH,7)=CMBAL(KRCH,7)+WATBAL(7)*THYINS
      CMBAL(KRCH,8)=CMBAL(KRCH,8)+WATBAL(8)*THYINS
      CMBAL(KRCH,9)=CMBAL(KRCH,9)+WATBAL(9)*THYINS
      CMBAL(KRCH,10)=CMBAL(KRCH,10)+WATBAL(10)*THYINS
      CMBAL(KRCH,11)=CMBAL(KRCH,11)+WATBAL(11)*THYINS
      CMBAL(KRCH,12)=CMBAL(KRCH,12)+WATBAL(16)*THYINS
  600 CONTINUE
      WRITE(5,5000) KRCH,TTHR
 5000 FORMAT (1X,'WATBAL',5X,I2,5X,F8.4)
      WRITE(5,5001) (WATBAL(N3),N3=1,16)
 5001 FORMAT (1X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6,5X,F12.6)
      WRITE(5,5014) KRCH
 5014 FORMAT (1X,'CMBAL',5X,I2)
      WRITE(5,5015) (CMBAL(KRCH,N4),N4=1,12)
 5015 FORMAT (1X,E12.4,5X,E12.4,5X,E12.4,5X,E12.4,5X,E12.4)
  810 CONTINUE
      IF(IREP(KRCH,7).EQ.2) GO TO 830
      CHSTAT(1)=DEPTH(KRCH,1)
      D21=AREA(DEPTH(KRCH,1))*VEL(KRCH,1)
      CHSTAT(2)=D21
      DUM2=RCHPR(KRCH,2)+0.5
      N5=IFIX(DUM2)
      CHSTAT(3)=DEPTH(KRCH,N5)
      CHSTAT(4)=DBCPR(KRCH,4)

      WRITE(6,5002) KRCH,TTHR
 5002 FORMAT (1X,I2,5X,F8.4)
      WRITE(6,5003)(CHSTAT(N6),N6=1,4)
 5003 FORMAT(1X,4(F8.3,5X))
  830 CONTINUE
      IF(IREP(KRCH,4).EQ.2) GO TO 850
      FD1(1)=DBCPR(KRCH,26)
      FD1(2)=DBCPR(KRCH,27)
      FD1(3)=DBCPR(KRCH,28)
      FD1(4)=DBCPR(KRCH,29)
      FD1(5)=DBCPR(KRCH,9)
      FD1(6)=DBCPR(KRCH,23)
      FD1(7)=DBCPR(KRCH,24)
      FD1(8)=DBCPR(KRCH,5)
      WRITE(7,5006) KRCH,TTHR
 5006 FORMAT (1X,I2,5X,F8.4)
      WRITE(7,5007) (FD1(N4),N4=1,8)
 5007 FORMAT(1X,F8.3,5X,F8.3,5X,F8.3,5X,F8.3)
  850 CONTINUE
C     REPORT4 (UNIT=8) IS PRESENTLY INACTIVE.  ONLY ONE FARM-TURNOUT
C     IS PERMITTED IN THE DESCRIPTION OF DBC TYPE "1" STRUCTURE.
      IF(IREP(KRCH,5).EQ.2) GO TO 860
      FD2(1)=DBCPR(KRCH,40)
      FD2(2)=DBCPR(KRCH,44)
      FD2(3)=DBCPR(KRCH,43)
      FD2(4)=DBCPR(KRCH,47)
      FD2(5)=DBCPR(KRCH,51)
      FD2(6)=DBCPR(KRCH,48)
      FD2(7)=DBCPR(KRCH,49)
      FD2(8)=DBCPR(KRCH,50)
      WRITE(8,5008) KRCH,TTHR
 5008 FORMAT (1X,I2,5X,F8.4)
      WRITE(8,5009) (FD2(N7),N7=1,8)
 5009 FORMAT (1X,F8.3,5X,F8.3,5X,F8.3,5X,F8.3)
  860 CONTINUE
      IF(IREP(KRCH,3).EQ.2) GO TO 880
      CHK(1)=DBCPR(KRCH,15)
      CHK(2)=DBCPR(KRCH,10)
      CHK(3)=DBCPR(KRCH,18)
      CHK(4)=DBCPR(KRCH,19)
      CHK(5)=DBCPR(KRCH,20)
      WRITE(9,5010) KRCH,TTHR
 5010 FORMAT (1X,I2,5X,F8.4)
      WRITE(9,5011) (CHK(N8),N8=1,5)
 5011 FORMAT (1X,5(F8.3,5X))
  880 CONTINUE

      IF (IREP(KRCH,11).EQ.2) GO TO 890
      WRITE(12,2543) KRCH,TTHR
 2543 FORMAT(1X,I2,5X,F8.4)
      I20=DBCPR(KRCH,1)+1
      WRITE (12,5012) (DBCPR(KRCH,I10),I10=2,I20)
 5012 FORMAT (1X,F12.9,5X,F12.9,5X,F12.9,5X,F12.9,5X,F12.9,5X)
  890 CONTINUE
      IF (IREP(KRCH,12).EQ.2) GO TO 895
      WRITE(13,2543) KRCH,TTHR
      I21=UBCPR(KRCH,1)+1
      WRITE (13,5013) (UBCPR(KRCH,I11),I11=2,I21)
 5013 FORMAT (1X,F12.9,5X,F12.9,5X,F12.9,5X,F12.9,5X,F12.9,5X)
  895 CONTINUE
      IF(IREP(KRCH,6).EQ.2) GO TO 540
      DUM4=RCHPR(KRCH,2)+0.5
      I10=IFIX(DUM4)
      WRITE(10,4200) KRCH,TTHR,(RCHPR(KRCH,I1),I1=1,3)
 4200 FORMAT (1X,I2,5X,F8.4,5X,F8.2,5X,F8.4,5X,F8.4)
      WRITE(10,4201) (ADEPT(KRCH,I9),I9=1,I10)
 4201 FORMAT (1X,F6.3,5X,F6.3,5X,F6.3,5X,F6.3,5X,F6.3)
      WRITE(10,4202) (AVEL(KRCH,I9),I9=1,I10)
 4202 FORMAT (1X,F6.3,5X,F6.3,5X,F6.3,5X,F6.3,5X,F6.3)
  540 CONTINUE
```

```
      RETURN
      END
C
C
C
C
C
      SUBROUTINE OPERAT(TNOP)
C
C     THIS SUBROUTINE PERFORMS THE UPDATE OF UBCPR AND DBCPR MATRICES
C     AS OPERATION OF THE CANAL IS INITIALIZED BY SPECIFYING THE
C     REACH BEING OPERATED AND THE DURATION OF THE OPERATING CYCLE.
C     IT ALSO RECORDS THE REAL TIME THROUGH THE 'TIME' SUBROUTINE.
C
      COMMON /CIQ/IQ
      COMMON /CIREP/IREP(10,12)
      COMMON /CKRCH/KRCH
      COMMON /CCTTHR/CTTHR
      COMMON /CTDAY/TDAY
      COMMON /CATDAY/ATDAY
      COMMON /CNDAY/NDAY
      IQ=1
  160 CONTINUE

      PRINT 3100
 3100 FORMAT (/,1X,'OPERATIONS ARE NOW PERMITTED.')
      CALL TIME
      PRINT 3109
 3109 FORMAT(/,1X,'IF YOU WISH TO OPERATE, INPUT THE NO. 1;')
      PRINT 3010
 3010 FORMAT(/,1X,'OTHERWISE, INPUT THE NO. 999.')
      READ *,ABC
      ABC=ABC
      IF(ABC.GT.998) GO TO 170
      PRINT 3101
 3101 FORMAT (/,1X,'INPUT THE NUMBER OF THE REACH TO BE EXAMINED')
      PRINT 3102
 3102 FORMAT (1X,'AND/OR OPERATED.',1X,'INPUT THE NO. AS AN INTEGER.')
      READ *,NNRCH
      NNRCH=NNRCH
      KRCH=NNRCH
      CALL UBC
      CALL DBC
      IF(IREP(NNRCH,8).EQ.2) GO TO 160
      WRITE(11,3104) CTTHR,NDAY,ATDAY,NNRCH
 3104 FORMAT (1X,'CUM. HRS.=',F6.2,5X,'DAY=',I2,5X,'TIME=',F8.2,5X
     1,'RCH NO. =',I2)
```

```
      GO TO 160
  170 CONTINUE
      PRINT 3105
 3105 FORMAT (/,1X,'INPUT THE DURATION IN HOURS TILL THE NEXT REVIEW')
      PRINT 3106
 3106 FORMAT (1X,'AND OPERATION CYCLE,(AS A "REAL" NO.).')
      PRINT 3107
 3107 FORMAT (1X,'IF YOU WISH TO TERMINATE THE SIMULATION AT THIS')
      PRINT 3108
 3108 FORMAT (1X,'TIME, INPUT THE NUMBER "0.0".')
      READ *,TNOP
      TNOP=TNOP
      IF (TNOP.LT.0.01) GO TO 180
      RETURN
  180 CONTINUE
      STOP
      END
C
C
C
C
C
      SUBROUTINE TIME
C
C     THIS SUBROUTINE KEEPS TRACK OF THE REAL TIME AND WILL RECORD
C     THE NUMBER IF DAYS SINCE SIMULATION STARTED AND THE ACTUAL
C     TIME OF THE DAY.
C
      COMMON /CCTTHR/CTTHR
      COMMON /CTDAY/TDAY
      COMMON /CATDAY/ATDAY
      COMMON /CNDAY/NDAY
      DUM1=(CTTHR+12.0)/24.0 +0.50001
      NDAY=IFIX(DUM1)
      ADAY=FLOAT(NDAY)
      ATDAY=(CTTHR-(ADAY-1)*24.0)+TDAY
      PRINT 3200,NDAY,ATDAY
 3200 FORMAT(1X,'DAY=',I2,10X,'TIME=',F10.2)
      RETURN
      END
C
C
C
C
C
      SUBROUTINE HYDROL
```

```
C
C     THIS SUBROUTINE IS PARTIALLY COMPLETE. THE 'NESTED' SUBROUTINES
C     ROUGH, EVAP, PRECIP, SEEP, ODST ARE AT THE PRESENT TIME
C     UNAVAILABLE. THIS IMPLIES THAT A '1' SHOULD BE ENTERED FOR
C     ICHID X,2,3,4,5,6,7 IN THE ICHID MATRIX, WHICH IS CREATED BY
C     THE 'RCHDAT' PROGRAM.
C     IT WILL PRESENTLY ONLY UPDATE THE MINIMUM ALLOWABLE FLOW TO THE
C     DOWNSTREAM REACH(ES).
C
      COMMON /CICHID/ICHID(10,10)
      COMMON /CKRCH/KRCH
      COMMON /CQLAT/QLAT(10)
      COMMON /CQSEEP/QSEEP(10,20)
      COMMON /CQPREC/QPREC(10,20)
      COMMON /CQEVAP/QEVAP(10,20)
      COMMON /CQODST/QODST(10,20)
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CRCHPR/RCHPR(10,5)
      I7=KRCH
      QACUM=0.0
      IF(ICHID(I7,2).EQ.1) GO TO 210
      CALL ROUGH(I7)
  210 CONTINUE

      IF(ICHID(I7,3).EQ.1) GO TO 200
      IF(ICHID(I7,6).EQ.1) GO TO 220
      CALL EVAP(I7)
      QACUM=QACUM-QEVAP(I7,3)
  220 CONTINUE
      IF(ICHID(I7,5).EQ.1) GO TO 230
      CALL PRECIP(I7)
      QACUM=QACUM+QPREC(I7,3)
  230 CONTINUE
      IF(ICHID(I7,4).EQ.1) GO TO 240
      CALL SEEP(I7)
      QACUM=QACUM-QSEEP(I7,3)
  240 CONTINUE
      IF(ICHID(I7,7).EQ.1) GO TO 250
      CALL ODST(I7)
      QACUM=QACUM+QODST(I7,3)
  250 CONTINUE
      QLAT(I7)=QACUM
  200 CONTINUE
      RETURN
      END
C
C
```

```
C
      SUBROUTINE ROUGH(I7)
      RETURN
      END
C
C
C
      SUBROUTINE EVAP(I7)
      RETURN
      END
C
C
C
      SUBROUTINE PRECIP(I7)
      RETURN
      END
C
C
C
      SUBROUTINE SEEP(I7)
C
C     THIS SUBROUTINE WILL ASSIGN A NEW VALUE TO QSEEP ACCORDING
C     TO THE PRESENT DEPTH.  USING THE MORITZ FORMULA(METRIC), THE

C     RATE OF SEEPAGE LOSSES IS CONSTANT OVER THE TOTAL LENGTH OF
C     THE REACH.   (SECOND LEVEL SUBROUTINE)
C
      COMMON /CRCHPR/RCHPR(10,5)
      COMMON /CDEPTH/DEPTH(10,21)
      COMMON /CQSEEP/QSEEP(10,20)
      DUM1=RCHPR(I7,2)+0.5
      L=IFIX(DUM1)
      TAREA=0.0
      DO 100 I=1,L
      TAREA=TAREA+AREA(DEPTH(I7,I))
  100 CONTINUE
      AAREA=TAREA/L
      QSEEP(I7,3)=(1.155/100000)*QSEEP(I7,4)*AAREA**0.5
C     !!! QSEEP(I7,4) IS THE SEEPAGE COEFFICIENT IN IMPERIAL UNITS. !!!
      RETURN
      END
C
C
C
      SUBROUTINE ODST(I7)
      RETURN
      END
```

```
C
C
C
C
C
      SUBROUTINE HYDRT(GRAV,WFACTA,WFACTB,THYINS,RESMAX)
C
C     THIS SUBROUTINE WILL PERFORM THE SOLUTION OF FINITE DIFFERENCE
C     EQUATIONS USED IN THE HYDRAULIC ROUTING CYCLE. (AMEIN, FANG AND
C     MANZ)
C     FCOEF & GCOEF MATRICES ARE CREATED AND COMBINED WITH UPSTREAM
C     AND DOWNSTREAM BOUNDARY EQUATIONS AN 'U'-MATRIX IF FORMED.
C     THE NESTED SUBROUTINE 'GAUSEL' WILL SOLVE THE MATRIX AND
C     RESULTS ARE ASSIGNED TO ARRAY 'T'.
C
      COMMON /CGEOM/GEOM(10,21)
      COMMON /CKRCH/KRCH
      COMMON /CELEV/ELEV(10,21)
      COMMON /CROUGH/ROUGH(10,20)
      COMMON /CQLAT/QLAT(10)
      COMMON /CUBCPR/UBCPR(10,50)
      COMMON /CDBCPR/DBCPR(10,60)
      COMMON /CDEPTH/DEPTH(10,21)

      COMMON /CVEL/VEL(10,21)
      COMMON /CADEPT/ADEPT(10,21)
      COMMON /CAVEL/AVEL(10,21)
      COMMON /CIREP/IREP(10,12)
      COMMON /CIQ/IQ
      COMMON /CU/U(42,43)
      COMMON /CT/T(42)
      COMMON /CRCHPR/RCHPR(10,5)
      DIMENSION FNA00(21),FNA10(21)
      DIMENSION FNB00(21),FNB10(21),FNP00(21),FNP10(21),FDA00(21)
      DIMENSION FDB00(21),FDB10(21),FDA10(21),FDP00(21),FDP10(21)
      DIMENSION SL00(21),SL10(21)
      DIMENSION C1(21),C2(21),C3(21),C4(21),C5(21),C6(21),C7(21)
      DIMENSION C8(21),C9(21),C10(21),C11(21),C12(21),C13(21)
      DIMENSION C14(21),C15(21),C16(21),FCOEF(21,5),GCOEF(21,5)
      IQ=3
      DUM1=RCHPR(KRCH,2)+0.5
      L=IFIX(DUM1)
      L1=2*L
      L2=L1+1
      L3=L-1
      L4=2*L3
      L5=L1-1
      DO 270 J5=1,L3
      J8=J5+1
      FNA00(J5)=AREA(DEPTH(KRCH,J5))
      FNA10(J5)=AREA(DEPTH(KRCH,J8))
      FNB00(J5)=TOPW(DEPTH(KRCH,J5))
      FNB10(J5)=TOPW(DEPTH(KRCH,J8))
      FNP00(J5)=WETP(DEPTH(KRCH,J5))
      FNP10(J5)=WETP(DEPTH(KRCH,J8))
      FDA00(J5)=DAREA(DEPTH(KRCH,J5))
      FDA10(J5)=DAREA(DEPTH(KRCH,J8))
      FDB00(J5)=DTOPW(DEPTH(KRCH,J5))
      FDB10(J5)=DTOPW(DEPTH(KRCH,J8))
      FDP00(J5)=DWETP(DEPTH(KRCH,J5))
      FDP10(J5)=DWETP(DEPTH(KRCH,J8))
      SL00(J5)=VEL(KRCH,J5)*VEL(KRCH,J5)*ROUGH(KRCH,2)*ROUGH(KRCH,2)/((F
     1NA00(J5)/FNP00(J5))**1.33333)
      SL10(J5)=VEL(KRCH,J8)*VEL(KRCH,J8)*ROUGH(KRCH,2)*ROUGH(KRCH,2)/((F
     1NA10(J5)/FNP10(J5))**1.33333)
      C1(J5)=-(DEPTH(KRCH,J8)+DEPTH(KRCH,J5))
      C2(J5)=WFACTB*((FNA10(J5)/FNB10(J5))+(FNA00(J5)/FNB00(J5)))
      C3(J5)=WFACTB*(VEL(KRCH,J8)-VEL(KRCH,J5))
      C4(J5)=RCHPR(KRCH,4)*C2(J5)*C3(J5)
      C5(J5)=WFACTB*(VEL(KRCH,J8)+VEL(KRCH,J5))

      C6(J5)=WFACTB*(DEPTH(KRCH,J8)-DEPTH(KRCH,J5))
      C7(J5)=RCHPR(KRCH,4)*C5(J5)*C6(J5)
      C8(J5)=THYINS*QLAT(KRCH)*WFACTB*((1.0/FNB10(J5))+(1.0/FNB00(J5)))
      C9(J5)=-(VEL(KRCH,J8)+VEL(KRCH,J5))/RCHPR(KRCH,4)/GRAV
      C10(J5)=C5(J5)
      C11(J5)=WFACTB*(VEL(KRCH,J8)-VEL(KRCH,J5))
      C12(J5)=WFACTB*WFACTB*((VEL(KRCH,J8)*VEL(KRCH,J8))-(VEL(KRCH,J5)*V
     1EL(KRCH,J5)))/GRAV
      C13(J5)=2*C6(J5)
      C14(J5)=2*(ELEV(KRCH,J8)-ELEV(KRCH,J5))
      C15(J5)=RCHPR(KRCH,3)*WFACTB*(SL10(J5)+SL00(J5))
      C16(J5)=QLAT(KRCH)*RCHPR(KRCH,3)*WFACTB*((VEL(KRCH,J8)/
     1AREA(DEPTH(KRCH,J8)))+(VEL(KRCH,J5)/AREA(DEPTH(KRCH,J5))))/GRAV
  270 CONTINUE
      MAX=21
      LSAFE=0
  280 CONTINUE
      LSAFE=LSAFE+1
      IF(LSAFE.LE.MAX) GO TO 290
      CALL TIME
      PRINT 6000,KRCH
 6000 FORMAT (/,1X,'APPARANT NUMERICAL INSTABILITY REACH NO.',I2)
      PRINT 6001,MAX
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6001 FORMAT (1X,'MAXIMUM NO. OF ITERATIONS ALLOWED IS:',1X,I2)
      PRINT 6002
6002 FORMAT (1X,'IF YOU WISH TO TERMINATE THIS SIMULATION,')
      PRINT 6003
6003 FORMAT (1X,'INPUT THE INTEGER, "999".')
      PRINT 6004
6004 FORMAT (1X,'IF YOU WISH TO INCREASE THE MAXIMUM NUMBER OF')
      PRINT 6005
6005 FORMAT (1X,'ITERATIONS, INPUT,WHEN PROMPTED, THE NUMBER OF')
      PRINT 6006
6006 FORMAT (1X,'EXTRA ITERATIONS YOU WOULD LIKE PERFORMED.  THIS')
      PRINT 6007
6007 FORMAT (1X,'NUMBER MUST ALSO BE AN INTEGER.')
      READ *,MMAX
      IF (MMAX.GE.20) GO TO 300
      MAX=MAX+MMAX
 290  CONTINUE
      IT=0
      DO 310 J5=1,L3
      J8=J5+1
      IT=IT+1
      FNA01=AREA(ADEPT(KRCH,J5))
      FNA11=AREA(ADEPT(KRCH,J8))

      FNB01=TOPW(ADEPT(KRCH,J5))
      FNB11=TOPW(ADEPT(KRCH,J8))
      FNP01=WETP(ADEPT(KRCH,J5))
      FNP11=WETP(ADEPT(KRCH,J8))
      FDA01=DAREA(ADEPT(KRCH,J5))
      FDA11=DAREA(ADEPT(KRCH,J8))
      FDB01=DTOPW(ADEPT(KRCH,J5))
      FDB11=DTOPW(ADEPT(KRCH,J8))
      FDP01=DWETP(ADEPT(KRCH,J5))
      FDP11=DWETP(ADEPT(KRCH,J8))
      SL01=AVEL(KRCH,J5)*AVEL(KRCH,J5)*ROUGH(KRCH,2)*ROUGH(KRCH,2)/((FNA
     101/FNP01)**1.33333)
      SL11=AVEL(KRCH,J8)*AVEL(KRCH,J8)*ROUGH(KRCH,2)*ROUGH(KRCH,2)/((FNA
     111/FNP11)**1.33333)
      D1=ADEPT(KRCH,J8)+ADEPT(KRCH,J5)
      D2=(FNA11/FNB11)+(FNA01/FNB01)
      D3=AVEL(KRCH,J8)-AVEL(KRCH,J5)
      D4=AVEL(KRCH,J8)+AVEL(KRCH,J5)
      D5=ADEPT(KRCH,J8)-ADEPT(KRCH,J5)
      D6=(1.0/FNB11)+(1.0/FNB01)
      D7=(AVEL(KRCH,J8)*AVEL(KRCH,J8))-(AVEL(KRCH,J5)*AVEL(KRCH,J5))
      D8=SL11+SL01
      D9=(AVEL(KRCH,J8)/FNA11)+(AVEL(KRCH,J5)/FNA01)
      D10=RCHPR(KRCH,4)
      D11=RCHPR(KRCH,3)
      FCOEF(J5,5)=D1+C1(J5)+(D10*WFACTA*WFACTA*D2*D3)+(D10*WFACTA*C2(J5)
     1*D3)+(D10*WFACTA*C3(J5)*D2)+C4(J5)+(D10*WFACTA*WFACTA*D4*D5)+(D10*
     2WFACTA*C5(J5)*D5)+(D10*WFACTA*C6(J5)*D4)+C7(J5)-(THYINS*QLAT(KRCH)
     3*WFACTA*D6)-C8(J5)
      GCOEF(J5,5)=(D4/D10/GRAV)+C9(J5)+(WFACTA*WFACTA*D7/GRAV)+(C10(J5)*
     1WFACTA*D3/GRAV)+(C11(J5)*WFACTA*D4/GRAV)+C12(J5)+(2.0*WFACTA*D5)+C
     213(J5)+C14(J5)+(D11*WFACTA*D8)+C15(J5)+(QLAT(KRCH)*D11*WFACTA*D9/G
     3RAV)+C16(J5)
      FCOEF(J5,5)=-FCOEF(J5,5)
      GCOEF(J5,5)=-GCOEF(J5,5)
      D12=1.0-FNA01/FNB01/FNB01*FDB01
      D13=1.0-FNA11/FNB11/FNB11*FDB11
      D14=(FDP01/FNP01)-(FNB01/FNA01)
      D15=(FDP11/FNP11)-(FNB11/FNA11)
      D16=AVEL(KRCH,J5)
      D17=AVEL(KRCH,J8)
      D18=FDB01/FNB01/FNB01
      D19=FDB11/FNB11/FNB11
      D20=(FNA11/FNB11)+(FNA01/FNB01)
      FCOEF(J5,1)=1.0+(D10*WFACTA*WFACTA*D12*D3)+(D10*WFACTA*C3(J5)*D12
     1)-(D10*WFACTA*WFACTA*D4)-(D10*WFACTA*C5(J5))+(THYINS*QLAT(KRCH)*WF

     2ACTA*D18)
      FCOEF(J5,3)=1.0+(D10*WFACTA*WFACTA*D13*D3)+(D10*WFACTA*C3(J5)*D13)
     1+(D10*WFACTA*WFACTA*D4)+(D10*WFACTA*C5(J5))+(THYINS*QLAT(KRCH)*WF
     2ACTA*D19)
      FCOEF(J5,2)=(-D10*WFACTA*WFACTA*D20)-(D10*WFACTA*C2(J5))+(D10*WFAC
     1TA*WFACTA*D5)+(D10*WFACTA*C6(J5))
      FCOEF(J5,4)=(D10*WFACTA*WFACTA*D20)+(D10*WFACTA*C2(J5))+(D10*WFACT
     1A*WFACTA*D5)+(D10*WFACTA*C6(J5))
      GCOEF(J5,1)=(-2.0*WFACTA)+(4.0/3.0*D11*WFACTA*SL01*D14)-(QLAT(KRCH
     1)*D11*WFACTA*D16*FNB01/FNA01/FNA01/GRAV)
      GCOEF(J5,3)=(2.0*WFACTA)+(4.0/3.0*D11*WFACTA*SL11*D15)-(QLAT(KRCH)
     1*D11*WFACTA*D17*FNB11/FNA11/FNA11/GRAV)
      GCOEF(J5,2)=(1.0/GRAV/D10)-(2.0*WFACTA*WFACTA*D16/GRAV)+(C11(J5)*W
     1FACTA/GRAV)+(2.0*D11*WFACTA*SL01/D16)+(QLAT(KRCH)*D11*WFACTA/GRAV/
     2FNA01)-(C10(J5)*WFACTA/GRAV)
      GCOEF(J5,4)=(1.0/GRAV/D10)+(2.0*WFACTA*WFACTA*D17/GRAV)+(C10(J5)*W
     1FACTA/GRAV)+(C11(J5)*WFACTA/GRAV)+(2.0*D11*WFACTA*SL11/D17)+(Q
     2LAT(KRCH)*D11*WFACTA/GRAV/FNA11)
  310 CONTINUE
      CALL UBC
      CALL DBC
      IF(RESMAX.LT.ABS(UBCPR(KRCH,8))) GO TO 320
      IF(RESMAX.LT.ABS(DBCPR(KRCH,8))) GO TO 320
      DO 330 I8=1,L3
      IF(RESMAX.LT.ABS(FCOEF(I8,5))) GO TO 320
      IF(RESMAX.LT.ABS(GCOEF(I8,5))) GO TO 320
  330 CONTINUE
      WRITE(9,6008) KRCH,LSAFE
 6008 FORMAT(1X,I3,5X,I3)
      RETURN
  320 CONTINUE
      DO 340 I11=1,L2
      DO 350 I12=1,L1
      U(I12,I11)=0.0
  350 CONTINUE
  340 CONTINUE
      U(1,1)=UBCPR(KRCH,6)
      U(1,2)=UBCPR(KRCH,7)
      U(1,L2)=UBCPR(KRCH,8)
      U(L1,L2-2)=DBCPR(KRCH,6)
      U(L1,L2-1)=DBCPR(KRCH,7)
      U(L1,L2)=DBCPR(KRCH,8)
      DO 360 I13=1,L3
      I14=I13*2
      I15=I14-1
      I16=I14+2

      I17=1
      U(I14,L2)=FCOEF(I13,5)
      DO 370 I18=I15,I16
      U(I14,I18)=FCOEF(I13,I17)
      I17=I17+1
  370 CONTINUE
  360 CONTINUE
      DO 380 J1=1,L3
      J2=2*J1+1
      J3=J2-2
      J4=J2+1
      J10=1
      U(J2,L2)=GCOEF(J1,5)
      DO 390 J6=J3,J4
      U(J2,J6)=GCOEF(J1,J10)
      J10=J10+1
  390 CONTINUE
  380 CONTINUE
      CALL GAUSEL(L)
      DO 400 J7=1,L
      J11=J7*2
      J9=J11-1
      ADEPT(KRCH,J7)=T(J9)+ADEPT(KRCH,J7)
      AVEL(KRCH,J7)=T(J11)+AVEL(KRCH,J7)
  400 CONTINUE
      DO 410 J12=1,L
      IF(ADEPT(KRCH,J12).LE.0.000) GO TO 420
  410 CONTINUE
      GO TO 280
  420 CONTINUE
      CALL TIME
      PRINT 4010,J12,KRCH
 4010 FORMAT (/,1X,'DEPTH AT NODE,',5X,I2,5X,'OF REACH,',5X,I2,5X,'IS LE
     1SS THAN 0.001 METER.')
      PRINT 4011
 4011 FORMAT (1X,'SIMULATION ENDS.')
  300 CONTINUE
      STOP
      END
C
C
C
C
C
      SUBROUTINE GAUSEL(L)
C
C     THIS SUBROUTINE WILL PERFORM THE GAUSSEL ELEMINATION PROCEDURE
C     TO OBTAIN THE RESULTING 'T'-ARRAY.
C
      COMMON /CU/U(42,43)
      COMMON /CT/T(42)
      COMMON /CKRCH/KRCH
      L1=2*L
      L2=L1+1
      L5=L1-1
      DO 430 M1=1,L5
      DO 440 M2=M1,L1
      IF(ABS(U(M2,M1)).GE.0.0000001) GO TO 450
  440 CONTINUE
  460 CONTINUE
      PRINT 4100
 4100 FORMAT('1','ROUTING PROBLEM IN GAUSSIAN ELIMINATION ROUTINE')
      PRINT 4101,KRCH
 4101 FORMAT (1X,'IN REACH NUMBER,',5X,I2,'.')
      PRINT 4102
 4102 FORMAT (1X,'SIMULATION ENDS.')
      GO TO 470
  450 CONTINUE
      IF(M2.EQ.M1)GO TO 480
      DO 490 M3=1,L2
      B2=U(M1,M3)
      U(M1,M3)=U(M2,M3)
      U(M2,M3)=B2
  490 CONTINUE
  480 CONTINUE
      M4=M1+1
      DO 500 M5=M4,L1
      B3=U(M5,M1)/U(M1,M1)
      DO 510 M6=M1,L2
      U(M5,M6)=U(M5,M6)-B3*U(M1,M6)
  510 CONTINUE
      IF(ABS(U(M5,M5)).LE.0.000000001) GO TO 460
  500 CONTINUE
  430 CONTINUE
      T(L1)=U(L1,L2)/U(L1,L1)
      DO 520 M7=1,L5
      M8=L1-M7
      B4=0.0
      M10=M8+1
      DO 530 M11=M10,L1
      B4=B4+U(M8,M11)*T(M11)
  530 CONTINUE

      T(M8)=(U(M8,L2)-B4)/U(M8,M8)
  520 CONTINUE
      RETURN
  470 CONTINUE
      STOP
      END
```

APPENDIX I

LISTING OF COMPUTER PROGRAM WHICH SETS UP DATA FILE WHICH CONTROLS INDIVIDUAL RUNS OF THE ICSS MODEL

```
      PROGRAM SIMDAT
      OPEN (UNIT=1,FILE='SIMDAT1',STATUS='NEW')
      PRINT *, 'WELCOME TO SIMDAT'
      PRINT *, 'INTERACTIVE CREATION OF SIMULATION DATA FILE'
      PRINT *, 'PLEASE INPUT DATA AS REQUESTED.'
999   CONTINUE
      PRINT 6001
6001  FORMAT (/,'INPUT THE DURATION OF SIMULATION IN HOURS.')
      READ *,TSTUD
      WRITE (1,6002)TSTUD
6002  FORMAT (1X,F8.4)
      PRINT *, 'INPUT THE TIME OF DAY THE SIMULATION STARTS.'
      PRINT *, 'EG. ONE THIRTY P.M. = 13.50'
      READ *,TDAY
      WRITE (1,6004)TDAY
6004  FORMAT (1X,F6.2)
      PRINT *, 'INPUT THE DURATION OF THE HYDRAULIC ROUTING TIME INCREME
     +NT IN HOURS.'
      READ *,THYINH
      WRITE (1,6005)THYINH
6005  FORMAT (1X,F8.4)
      PRINT *, 'INPUT THE DURATION OF THE HYDROLOGICAL TIME INCREMENTS I
     +N HOURS.'

      READ *,THYDRO
      WRITE (1,6003)THYDRO
6003  FORMAT (1X,F8.4)
      PRINT *, 'INPUT THE TOTAL NO. OF REACHES IN CANAL STUDIED.'
      READ *,NREACH
      WRITE (1,6006)NREACH
6006  FORMAT (1X,I2)
      PRINT *, 'INPUT THE WEIGHTING FACTOR USED IN HYDRAULIC ROUTING.'
      READ *,WFACTA
      WRITE (1,6007)WFACTA
6007  FORMAT (1X,F4.2)
      PRINT *, 'INPUT (1-WEIGHTING FACTOR)'
      READ *,WFACTB
      WRITE (1,6008)WFACTB
6008  FORMAT (1X,F4.2)
      PRINT *, 'INPUT THE MAXIMUM VALUE OF RESIDUAL IN HYDRAULIC ROUTING
     + SUBROUTINE.'
      READ *,RESMAX
      WRITE (1,6009)RESMAX
6009  FORMAT (1X,F5.4)
      PRINT *, 'INPUT THE GRAVITATIONAL CONSTANT.'
      READ *,GRAV
      WRITE (1,6010)GRAV
```

```
6010 FORMAT (1X,F10.5)
     REWIND (1)
     CLOSE (1)
     PRINT 6011
6011 FORMAT (//,'VERIFICATION OF DATA.')
     PRINT *, 'XXXXXXXXXXXXXXXXXXXXXXXXXXXXX'
     OPEN (UNIT=1,FILE='SIMDAT1',STATUS='OLD')
     READ (1,6002)TSTUD
     PRINT 6012,TSTUD
6012 FORMAT ('TSTUD= ',F8.4)
     READ (1,6004)TDAY
     PRINT 6014,TDAY
6014 FORMAT ('TDAY= ',F6.2)
     READ (1,6005)THYINH
     PRINT 6015,THYINH
6015 FORMAT ('THYINH= ',F8.4)
     READ (1,6003)THYDRO
     PRINT 6013,THYDRO
6013 FORMAT ('THYDRO= ',F8.4)
     READ (1,6006)NREACH
     PRINT 6016,NREACH
6016 FORMAT ('NREACH= ',I2)
     READ (1,6007)WFACTA

     PRINT 6017,WFACTA
6017 FORMAT ('WFACTA= ',F4.2)
     READ (1,6008)WFACTB
     PRINT 6018,WFACTB
6018 FORMAT ('WFACTB= ',F4.2)
     READ (1,6009)RESMAX
     PRINT 6019,RESMAX
6019 FORMAT ('RESMAX= ',F5.4)
     READ (1,6010)GRAV
     PRINT 6020,GRAV
6020 FORMAT ('GRAV= ',F10.5)
     PRINT 6021
6021 FORMAT (/,1X,'CHECK DATA FOR ERRORS')
     PRINT *, 'IF OK., INPUT THE NUMBER 1'
     PRINT *, 'IF THERE ARE ANY ERRORS, INPUT THE NUMBER 999'
     READ *, ERROR
     IF (ERROR.GT.1) GO TO 999
     PRINT *, 'DATA IS CORRECT AND WILL BE PLACED IN FILE: SIMDAT1'
     REWIND (1)
     CLOSE (1)
     STOP
     END
```

APPENDIX J

LISTING OF COMPUTER PROGRAM WHICH SETS UP DATA FILE WHICH CONTAINS INPUT DATA ON EACH REACH OF THE CANAL TO BE SIMULATED BY THE ICSS MODEL.

```
      PROGRAM RCHDAT
      DIMENSION RCHPR(5),ICHID(10),ELEV(21),GEOM(21),ROUGH(20)
      DIMENSION QSEEP(20),QPREC(20),QODST(20),QEVAP(20),UBCPR(60)
      DIMENSION DBCPR(60),IREP(12)
      OPEN (UNIT=2,FILE='RCHDAT1',STATUS='NEW')
      PRINT *, 'WELCOME TO PROGRAM RCHDAT.'
      PRINT *, 'INTERACTIVE CREATION OF REACH DATA FILE.'
      PRINT *, 'PLEASE INPUT DATA AS REQUESTED.'
      PRINT 4999
 4999 FORMAT (//,'INPUT THE TOTAL NUMBER OF REACHES.')
      READ *,NREACH
      WRITE (2,4998) NREACH
 4998 FORMAT (1X,I2)
      DO 300 L=1,NREACH
   10 CONTINUE
      PRINT 5000
 5000 FORMAT (//,'INPUT THE NUMBER OF THE REACH.')
      READ *,KRCH
      PRINT *, 'INPUT THE LENGTH OF THE REACH.'
      READ *,RCHPR(1)
      PRINT *, 'INPUT THE NO. OF DISTANCE NODES ALONG THE REACH.'
      READ *,RCHPR(2)

      PRINT *, 'INPUT THE FSL OF THE REACH.'
      READ *,RCHPR(5)
      PRINT *, 'INPUT THE UPSTREAM BOTTOM ELEVATION OF THE REACH.'
      READ *,ELEV(1)
      PRINT *, 'INPUT THE DOWNSTREAM BOTTOM ELEVATION OF THE REACH.'
      K4=IFIX(RCHPR(2)+0.5)
      READ *,ELEV(K4)
      PRINT 1000
 1000 FORMAT ('****************************************************')
      PRINT 1001
 1001 FORMAT ('********************VERIFICATION********************')
      PRINT 1002
 1002 FORMAT ('****************************************************')
      PRINT 100,KRCH
  100 FORMAT ('KRCH= ',I2)
      PRINT 101,RCHPR(1),RCHPR(2),RCHPR(5)
  101 FORMAT ('RCHPR(1)= ',F8.2,2X,'RCHPR(2)= ',F6.2,2X,'RCHPR(5)= ',
     +F6.2)
      PRINT 102,ELEV(1),ELEV(K4)
  102 FORMAT ('ELEV(1)= ',F8.2,2X,'ELEV(K4)= ',F8.2)
      PRINT 1003
 1003 FORMAT ('****************************************************')
      PRINT 1004
 1004 FORMAT ('IF ANY DATA MUST BE REENTERED,INPUT THE NUMBER 999, IF OK
     +., INPUT THE NUMBER 1')
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 10
      PRINT 1005
 1005 FORMAT ('INPUT DATA IS CORRECT AND WILL BE PLACED IN FILE: RCHDAT1
     +')
      WRITE (2,5001)KRCH
 5001 FORMAT (1X,I2)
      WRITE (2,5002)RCHPR(1),RCHPR(2),RCHPR(5)
 5002 FORMAT (1X,F8.2,2F6.2)
      WRITE (2,5005)ELEV(1),ELEV(K4)
 5005 FORMAT (1X,2F8.2)
      PRINT 1006
 1006 FORMAT ('------------------------------------------------------------')
   20 CONTINUE
      PRINT *, 'INPUT THE CHANNEL GEOMETRY COEFFICIENTS AS REQUESTED.'
      PRINT *, 'FORMAT FOR ALL GEOMETRY FUNCTIONS IS: F8.4'
      PRINT 1000
      PRINT *, 'INPUT THE FOUR AREA COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=1,4)
      PRINT *, 'INPUT THE FOUR TOP WIDTH COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=5,8)

      PRINT *, 'INPUT THE FOUR WETTED PERIMETER COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=9,12)
      PRINT *, 'INPUT THE THREE DA/DY COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=13,15)
      PRINT *, 'INPUT THE THREE DB/DY COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=16,18)
      PRINT *, 'INPUT THE THREE DP/DY COEFS. IN SEQUENCE.'
      READ *,(GEOM(I),I=19,21)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 103,(GEOM(I),I=1,4)
  103 FORMAT ('GEOM(1) = ',F8.4,2X,'GEOM(2) = ',F8.4,2X,'GEOM(3) = ',
     +F8.4,2X,'GEOM(4) = ',F8.4)
      PRINT 104,(GEOM(I),I=5,8)
  104 FORMAT ('GEOM(5) = ',F8.4,2X,'GEOM(6) = ',F8.4,2X,'GEOM(7) = ',
     +F8.4,2X,'GEOM(8) = ',F8.4)
      PRINT 105,(GEOM(I),I=9,12)
  105 FORMAT ('GEOM(9) = ',F8.4,2X,'GEOM(10)= ',F8.4,2X,'GEOM(11)= ',
     +F8.4,2X,'GEOM(12)= ',F8.4)
      PRINT 106,(GEOM(I),I=13,15)
  106 FORMAT ('GEOM(13)= ',F8.4,2X,'GEOM(14)= ',F8.4,2X,'GEOM(15)= ',
     +F8.4)
```

```
      PRINT 107,(GEOM(I),I=16,18)
  107 FORMAT ('GEOM(16)= ',F8.4,2X,'GEOM(17)= ',F8.4,2X,'GEOM(18)= ',
     +F8.4)
      PRINT 108,(GEOM(I),I=19,21)
  108 FORMAT ('GEOM(19)= ',F8.4,2X,'GEOM(20)= ',F8.4,2X,'GEOM(21)= ',
     +F8.4)
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 20
      PRINT 1005
      WRITE (2,5008) (GEOM(I),I=1,12)
 5008 FORMAT (1X,F8.4,2X,F8.4,2X,F8.4,2X,F8.4,2X)
      WRITE (2,5009) (GEOM(I),I=13,21)
 5009 FORMAT (1X,F8.4,2X,F8.4,2X,F8.4,2X)
      PRINT 1006
   90 CONTINUE
      PRINT *, 'INPUT THE ROUGHNESS INDEX.'
      READ *,ICHID(2)
      PRINT *, 'INPUT THE NO. OF ROUGHNESS PARAMETERS.'
      READ *,ROUGH(1)
      PRINT *, 'INPUT THE ROUGHNESS PARAMETERS ONE AT A TIME AS REQUESTE
     +D.'

      PRINT *, 'FORMAT FOR ALL PARAMETERS IS: F12.6'
      K8=IFIX(ROUGH(1)+0.5)+1
      READ *,(ROUGH(K9),K9=2,K8)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 109,ICHID(2)
  109 FORMAT ('ICHID(2)= ',I2)
      PRINT 110,ROUGH(1)
  110 FORMAT ('ROUGH(1)= ',F4.0)
      PRINT *, 'FOLLOWING ARE THE ROUGHNESS PARAMETERS.....'
      DO 111, K9=2,K8
      PRINT 112,ROUGH(K9)
  112 FORMAT (F12.6)
  111 CONTINUE
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 90
      PRINT 1005
      WRITE (2,5013)ICHID(2)
 5013 FORMAT (1X,I2)
      WRITE (2,5014)ROUGH(1)
```

```
5014 FORMAT (1X,F4.0)
     WRITE (2,5015) (ROUGH(K9),K9=2,K8)
5015 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
     PRINT 1006
  99 CONTINUE
     PRINT *, 'INPUT THE TOTAL LATERAL INFLOW TO THE REACH.'
     READ *,QLAT
     PRINT *, 'INPUT THE LATERAL FLOW INDEX FOR THE REACH.'
     READ *,ICHID(3)
     PRINT *, 'INPUT THE SEEPAGE INDEX FOR THE REACH.'
     READ *,ICHID(4)
     PRINT *, 'INPUT THE NO OF SEEPAGE PARAMETERS.'
     READ *,QSEEP(1)
     PRINT *, 'INPUT THE SEEPAGE PARAMETERS ONE AT A TIME AS REQUESTED
    +.'
     PRINT *, 'FORMAT FOR ALL PARAMETERS IS: F12.6'
     K10=IFIX(QSEEP(1)+0.5)+1
     READ *,(QSEEP(K11),K11=2,K10)
     PRINT 1000
     PRINT 1001
     PRINT 1002
     PRINT 113,QLAT
 113 FORMAT ('QLAT= ',F12.6)

     PRINT 114,ICHID(3),ICHID(4)
 114 FORMAT ('ICHID(3)= ',I2,2X,'ICHID(4)= ',I2)
     PRINT 115,QSEEP(1)
 115 FORMAT ('QSEEP(1)= ',F4.0)
     PRINT *, 'FOLLOWING ARE THE SEEPAGE PARAMETERS.....'
     DO 116, K11=2,K10
     PRINT 117,QSEEP(K11)
 117 FORMAT (F12.6)
 116 CONTINUE
     PRINT 1003
     PRINT 1004
     READ *,ICHK
     IF (ICHK.GT.1) GO TO 99
     PRINT 1005
     WRITE (2,5016)QLAT
5016 FORMAT (1X,F12.6)
     WRITE (2,5017)ICHID(3)
5017 FORMAT (1X,I2)
     WRITE (2,5018)ICHID(4)
5018 FORMAT (1X,I2)
     WRITE (2,5019)QSEEP(1)
5019 FORMAT (1X,F4.0)
     WRITE (2,5020) (QSEEP(K11),K11=2,K10)
 5020 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      PRINT 1006
   98 CONTINUE
      PRINT *, 'INPUT THE PPT INDEX FOR THE REACH.'
      READ *,ICHID(5)
      PRINT *, 'INPUT THE NO. OF  PPT  PARAMETERS.'
      READ *,QPREC(1)
      PRINT *, 'INPUT THE PPT PARAMETERS ONE AT A TIME AS REQUESTED.'
      PRINT *, 'FORMAT FOR ALL PPT PARAMETERS IS: F12.6'
      K12=IFIX(QPREC(1)+0.5)+1
      READ *,(QPREC(I),I=2,K12)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 118,ICHID(5)
  118 FORMAT ('ICHID(5)= ',I2)
      PRINT 119,QPREC(1)
  119 FORMAT ('QPREC(1)= ',F4.0)
      PRINT *, 'FOLLOWING ARE THE PPT PARAMETERS.....'
      DO 120 I=2,K12
      PRINT 121,QPREC(I)
  121 FORMAT (F12.6)
  120 CONTINUE

      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 98
      PRINT 1005
      WRITE (2,5021)ICHID(5)
 5021 FORMAT (1X,I2)
      WRITE (2,5022)QPREC(1)
 5022 FORMAT (1X,F4.0)
      WRITE (2,5023) (QPREC(I),I=2,K12)
 5023 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      PRINT 1006
  150 CONTINUE
      PRINT *, 'INPUT THE EVAPORATION INDEX FOR THE REACH.'
      READ *,ICHID(6)
      PRINT *, 'INPUT THE NO. OF EVAPORATION PARAMETERS.'
      READ *,QEVAP(1)
      PRINT *, 'INPUT THE EVAPORATION PARAMETERS ONE AT A TIME AS REQUES
     +TED.'
      PRINT *, 'FORMAT FOR ALL EVAPORATION PARAMETERS IS: F12.6'
      K15=IFIX(QEVAP(1)+0.5)+1
      READ *,(QEVAP(I),I=2,K15)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 122,ICHID(6)
  122 FORMAT ('ICHID(6)= ',I2)
      PRINT 123,QEVAP(1)
  123 FORMAT ('QEVAP(1)= ',F4.0)
      PRINT *, 'THE FOLLOWING ARE THE EVAPORATION PARAMETERS.....'
      DO 124 I=2,K15
      PRINT 125,QEVAP(I)
  125 FORMAT (F12.6)
  124 CONTINUE
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 150
      PRINT 1005
      WRITE (2,5024)ICHID(6)
 5024 FORMAT (1X,I2)
      WRITE (2,5025)QEVAP(1)
 5025 FORMAT (1X,F4.0)
      WRITE (2,5026) (QEVAP(I),I=2,K15)
 5026 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      PRINT 1006

  160 CONTINUE
      PRINT *, 'INPUT THE OTHER DIST. IN/OUT FLOW INDEX FOR THE REACH.'
      READ *,ICHID(7)
      PRINT *, 'INPUT THE NO. OF OTHER DIST. IN/OUT FLOW PARAMETERS.'
      READ *,QODST(1)
      PRINT *, 'INPUT THE NO. OF DIST. IN/OUT FLOW PARAMETERS ONE AT A T
     +IME AS REQUESTED.'
      PRINT *, 'FORMAT FOR ALL IN/OUT FLOW PARAMETERS IS: F12.6'
      K16=IFIX(QODST(1)+0.5)+1
      READ *,(QODST(I),I=2,K16)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 126,ICHID(7)
  126 FORMAT ('ICHID(7)= ',I2)
      PRINT 127,QODST(1)
  127 FORMAT ('QODST(1)= ',F4.0)
      PRINT *, 'FOLLOWING ARE THE OTHER DIST. IN/OUT FLOW PARAMETERS....
     +.'
      DO 128 I=2,K16
      PRINT 129,QODST(I)
  129 FORMAT (F12.6)
  128 CONTINUE
```

```
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 160
      PRINT 1005
      WRITE (2,5027)ICHID(7)
 5027 FORMAT (1X,I2)
      WRITE (2,5028)QODST(1)
 5028 FORMAT (1X,F4.0)
      WRITE (2,5029) (QODST(I),I=2,K16)
 5029 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      PRINT 1006
  180 CONTINUE
      PRINT *, 'INPUT THE UBC-TYPE INDEX FOR THE REACH.'
      READ *,ICHID(8)
      PRINT *, 'INPUT THE DBC-TYPE INDEX FOR THE REACH.'
      READ *,ICHID(9)
      PRINT *, 'INPUT THE NO. OF UBC PARAMETERS.'
      READ *,UBCPR(1)
      PRINT *, 'INPUT THE UBC PARAMETERS ONE AT A TIME AS REQUESTED.'
      PRINT *, 'FORMAT FOR ALL UBC PARAMETERS IS: F12.6'
      K18=IFIX(UBCPR(1)+0.5)+1
      READ *,(UBCPR(I),I=2,K18)

      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 130,ICHID(8),ICHID(9)
  130 FORMAT ('ICHID(8)= ',I2,2X,'ICHID(9)= ',I2)
      PRINT 131,UBCPR(1)
  131 FORMAT ('UBCPR(1)= ',F4.0)
      PRINT *, 'FOLLOWING ARE THE UBC PARAMETERS.....'
      PRINT 133,(UBCPR(I),I=2,K18)
  133 FORMAT (1X,F10.6,2X,F10.6,2X,F10.6,2X,F10.6,2X,F10.6,2X)
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 180
      PRINT 1005
      WRITE (2,5030)ICHID(8)
 5030 FORMAT (1X,I2)
      WRITE (2,5031)ICHID(9)
 5031 FORMAT (1X,I2)
      WRITE (2,5032)UBCPR(1)
 5032 FORMAT (1X,F4.0)
      WRITE (2,5033) (UBCPR(I),I=2,K18)
 5033 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
```

```
      PRINT 1006
  200 CONTINUE
      PRINT *, 'INPUT THE NO. OF DBC PARAMETERS.'
      READ *,DBCPR(1)
      PRINT *, 'INPUT THE DBC PARAMETERS ONE AT A TIME AS REQUESTED.'
      PRINT *, 'FORMAT FOR ALL DBC PARAMETERS IS: F12.6'
      K21=IFIX(DBCPR(1)+0.5)+1
      READ *,(DBCPR(I),I=2,K21)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT 134,DBCPR(1)
  134 FORMAT ('DBCPR(1)= ',F4.0)
      PRINT *, 'FOLLOWING ARE THE DBC PARAMETERS.....'
      PRINT 136,(DBCPR(I),I=2,K21)
  136 FORMAT (1X,F10.6,2X,F10.6,2X,F10.6,2X,F10.6,2X,F10.6,2X)
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1) GO TO 200
      PRINT 1005
      WRITE (2,5034)DBCPR(1)
 5034 FORMAT (1X,F4.0)

      WRITE (2,5035) (DBCPR(I),I=2,K21)
 5035 FORMAT (1X,F12.6,2X,F12.6,2X,F12.6,2X,F12.6,2X)
      PRINT 1006
  220 CONTINUE
      PRINT *, 'INPUT THE REPORT CONTROL INDICES ONE AT A TIME AS REQUES
     +TED.'
      PRINT *, 'FORMAT FOR EACH INDEX IS: I2'
      READ *,(IREP(J),J=1,12)
      PRINT 1000
      PRINT 1001
      PRINT 1002
      PRINT *, 'FOLLOWING ARE THE REPORT CONTROL INDICES.....'
      PRINT 137,(IREP(J),J=1,12)
  137 FORMAT (12(I2,2X))
      PRINT 1003
      PRINT 1004
      READ *,ICHK
      IF (ICHK.GT.1)GO TO 220
      PRINT 1005
      WRITE (2,5036) (IREP(J),J=1,12)
 5036 FORMAT (1X,12I2)
  300 CONTINUE
      PRINT *, 'FINAL DATA HAS BEEN ENTERED; DATA FILE IS CREATED.'
```

```
REWIND (2)
CLOSE (2)
PRINT 1000
PRINT 1006
STOP
END
```

APPENDIX K

OBSERVATIONS OF IRRIGATION CONVEYANCE SYSTEM EFFICIENCY IN SOUTHERN ALBERTA

C S A E
S C G R

Paper No. 84-308

OBSERVATIONS OF IRRIGATION CONVEYANCE SYSTEM EFFICIENCY IN SOUTHERN ALBERTA

by

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CANADIAN SOCIETY OF AGRICULTURAL ENGINEERING
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ABSTRACT

Observations of Irrigation Conveyance System Efficiency in Southern Alberta

Observations were made of the conveyance efficiency of a distributary canal typical of Southern Alberta. During a period of low farm irrigation water demand, water was released into the distributary canal at varying rates. Discharges were accurately measured at several locations along the length of the canal using sharp crested rectangular weirs. Considerable variation in conveyance efficiency was observed, not only between individual reaches, but also with discharge and the manner with which the discharges were changed. Explanations for the variations in conveyance efficiency are offered and the design of a future observation program is presented.

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OBSERVATIONS OF IRRIGATION CONVEYANCE SYSTEM EFFICIENCY IN SOUTHERN ALBERTA

by

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Department of Civil Engineering
University of Calgary

INTRODUCTION

Water losses from irrigation canals in Southern Alberta have been a subject for concern since the construction of the first irrigation conveyance systems in the area. This may be expected since water losses due to either seepage or spillage not only necessitate larger diversions from river systems, but also, frequently result in land adjacent irrigation canals being taken out of agricultural production due to water-logging and salinity.

Some of the earliest studies on water losses from irrigation canals were performed in the 1950's by the Prairie Farm Rehabilitation Administration, (PFRA), and are reported by Pohjakas and Rapp (1967). The study was directed toward the observation of seepage water losses from lined canals and dugouts. Their study employed the "ponding" method in which a reach of canal is isolated, filled with water and the rate of water level drop in the canal is observed. The principal objective of the study was evaluation of canal linings for which the ponding method is well suited.

The next major study in which canal water losses were observed and reported was performed by Alberta Environment commencing 1977 and continuing until 1980, Allison and Coles (1978), Allison and Elliot (1979) and McLeod and Coles (1980) and (1981). Water losses from several reaches of the main canal of the Lethbridge Northern Irrigation District were observed by measuring flow into and out of reaches of interest. (This is called the "inflow-outflow" method.) The canal, whose capacity is approximately 20 m^3/s, was in use while the observations were being made. The data reported from this study exhibits considerable variability even for water loss observations made when the canal was flowing at similar discharges. Unfortunately, insufficient information is presented with which to interpret the variations observed. The studies did, however, generate "rule-of-thumb" statistics which proved very useful for making decisions pertaining to the lining of the canal when it was rehabilitated.

The two studies presented, though suited for the purposes intended, do not provide information which can be used to predict actual water losses from operating irrigation canals in Southern Alberta. Ponding studies, at best, provide only relative information concerning water loss rates and their results are very difficult to extend to operating canals which usually contain several fixed and operable hydraulic structures, a variety of flow conditions, and are operating under unsteady flow conditions. Very few ponding studies report groundwater conditions adjacent the canals examined. The water loss study performed by Alberta Environment did not provide sufficient information which would permit interpretation of their observations. As indicated earlier, the study was quite sufficient for purposes intended but their results cannot be extended beyond the reaches examined, even on the same canal. On main canals in Southern Alberta, whose capacity frequently exceed 100 cubic meters per second, studies of individual reaches is justifiable since main canals represent only a small fraction of total canal length, (eg. several hundred kilometers). There are, however, several thousand kilometers of laterals and distributary canals which exhibit considerable diversity in terms of their physical, environmental and operational characteristics. Observation programs to observe water losses from individual canals cannot usually be justified.

Techniques are required which permit prediction of water losses from canals given specific and easily observed characteristics. Many empirical techniques are available for this purpose, Kraatz (1977), but these techniques are generally considered useful only when calibrated using observations made in the region in which they are to be applied. Before development of empirical methods to predict water losses from irrigation canals can proceed, techniques must be available to observe and interpret water loss rates from operating irrigation canals.

STUDY OBJECTIVES

The objectives of the study were as follows:

1. to observe the irrigation conveyance efficiency of an operating irrigation distribution canal, and
2. to determine the extent of observation program required to interpret conveyance efficiency observations.

The study was intended to be preliminary in nature.

DESCRIPTION OF STUDY CANAL

The canal chosen study is located in Southern Alberta approximately 20 km north-east of the city of Lethbridge. (See Figure 1). The canal itself forms part of the Lethbridge Northern Irrigation District and was constructed prior to 1920. The canal has a capacity at the inlet of about 1 m^3/s and is typical of older earth distribution canals found in the region.

The canal runs west to east and is approximately 9.0 km long. (See Figure 2). The total change in elevation is approximately 60 m. The canal incorporates 35 drop-structures, 17 farm turnouts of which 8 were active during the time of study, 12 check-drops and 6 culverts. There are 55 reaches separated by drop-structures, check-drops or culverts.

Generally, the canal had an irregular cross-section as shown in Figures 3 (a) to (f). The locations of these cross-sections are indicated in Figure 4. Recently cleaned or new reaches will develop irregularly shaped cross-sections in only a few years.

With the exception of the recently cleaned and new reaches, the canal was vegetated. Canal banks were covered with numerous grasses, forbes, shrubs and trees. Much of the canal was infested with emergent and submerged aquatic weeds and filamentous green algae. Severity of weed infestations varied considerably throughout canal length.

Analysis of soil samples, taken along the canal length at the time the water wells were drilled, reveal that the canal is constructed of soil material containing considerable clay and silt fractions, (50% to 98%) as shown in Table 1. Locations of the water wells which supplied the samples are shown in Figure 4.

Water wells drilled along the canal length suggest that the canal water surface and the adjacent ground water were related as shown in Figure 5. Several visible groundwater discharge areas adjacent to the canal support this conclusion.

Table 1. Analyses of Soil Samples

Well Number	Silt %	Clay %	Texture
1	48	29	clay loam
2	64	22	silt loam
3	53	45	silty clay
4	40	57	silty clay
5	70	21	silt loam
6	48	41	silty clay
7	58	22	silt loam
8	60	22	silt loam
9	41	57	silty clay
10	66	16	silt loam
11	49	26	silt loam
12	65	14	silt loam
13	60	37	silty clay loam

STUDY PROCEDURE

The inflow-outflow method, as described by Kraatz (1977), was used to observe water losses from the study canal. The canal was divided into five reaches, each bounded by a water measurement station, (WMS), as shown in Figure 4. Each WMS consisted of a rectangular sharp-crested weir, a stilling well and water level recorder, and a staff-gauge. Two 90° V-notch weirs were also installed to monitor farm water deliveries near stations three and five. All rectangular weirs were mounted on existing check-drop and drop structures. The rectangular and 90° V-notch weirs were calibrated prior to installation. Sketches showing the layout of each of installations are shown in Figures 6 (a) to (f).

The canal was monitored from August 28 to September 6, 1982. This study period was carefully chosen so as to avoid periods of high farm water demand. Flows during the high water demand period are near canal capacity and would have prohibited using existing structures to mount the rectangular weirs. Further, by performing the study during the low water demand period, flow into the canal could be altered without interfering with normal farm water delivery activities. Examination of farm water demand on the canal in previous years (from Lethbridge Northern Irrigation District records), suggested that the study be performed during late August to mid-September. The total farm water demand for the canal and the daily rainfall for the 1981 irrigation season are shown in Figure 7. (Rainfall data was collected at the Lethbridge Airport and reported in the 1981 Monthly Meteorological Summaries published by Environment Canada). No rainfall occurred during the course of the study.

Total farm water demand during the period of study varied between nil and sixty litres per second. Initially, water was being delivered at a turnout immediately downstream of WMS #3. This delivery lasted three days and was stopped; however, a water delivery immediately upstream of WMS #5 was started and lasted for six days. There was one large, unexpected and unrecorded delivery made immediately upstream of WMS #6 shortly after it was installed. This delivery was made by the farmer and not the canal operator. As well, a "small" delivery had been made for "several" days between WMS #4 and WMS #5. The canal operator had no idea this delivery was being made and was only casually announced to us by the farmer who was making it. Unfortunately, this was also not monitored, but conversation with the farmer suggested that it was only sufficient to water a small garden plot, (ie. less than five litres per second). All deliveries were stopped for two full days at the end of the study.

Water releases into the canal were made through a gated culvert from a lateral which had a capacity of approximately three cubic metres per second. There were no visible water measurement devices to assist the canal operator in making his deliveries. Releases of water into the study canal were determined solely by the number of turns of the wheel, irrespective of the depth of water in the lateral canal.

Water releases into the canal varied between sixty and one hundred and seventy litres per second. It was hoped that these releases could be held constant for periods of several days in order to allow the groundwater and canal flow conditions to achieve equilibrium conditions. Unfortunately, this was never achieved simultaneously throughout the entire length of the canal. This was primarily due to significant, unexpected and abrupt changes in the depth of water in the lateral canal, and, also because of changing farm water deliveries along the study canal itself.

RESULTS

Analyses of the data generated during the period of study is summarized in Table 2. Note that conveyance efficiency, as defined by the equation,

$$e_c = \frac{\text{Outflow from Reach}}{\text{Inflow to Reach}}$$

is highly variable in time and location, varying from 0.83 to 1.07 for individual reaches. The range of values 0.60 to 1.03, given for reach 1-6 are calculated based on the observed conveyance efficiencies of individual reaches. (As indicated previously, there were only two days in the entire study period in which no water deliveries were being made).

Table 2. Summary of Results

Reach	Length (m)	e_c
1 - 2	920	0.85 - 0.98
2 - 3	860	1.00 - 1.06
3 - 4	1570	0.90 - 1.00
4 - 5	1709	0.95 - 1.07
5 - 6	2398	0.83 - 0.93
1 - 6	7456	0.60 - 1.03

The upper values of e_c tended to occur immediately after the canal discharge was decreased, though reach 2 - 3 gained water during the entire period of study. Three reaches, 1 - 2, 3 - 4 and 5 - 6 consistently lost water but reach 3 - 4 did achieve an e_c value of 1.00 for brief periods of time. It is interesting to note that all land areas, apparently affected by groundwater discharge, were located along these three reaches.

The actual flow rates at WMS #1 and WMS #2 and the irrigation conveyance efficiency are shown in Figures 8 (a) to (d). Several details may be noted:

1. The efficiency of the reach varies from 0.85 to 0.96.
2. Higher flow rates exhibit greater loss rates but are very efficient. Conversely, lower flow rates, (eg. beginning of study), exhibit low loss rates but are less efficient.
3. e_c is at a maximum when low flows follow high flows.

Observation #1 is significant in that it is clearly apparent that conveyance efficiency is not constant but is a function of canal flow rate and canal environment.

Observation #2 is easily understood when it is noted that seepage losses are, at best, proportional to depth, whereas flow rates will increase proportional to water depth raised to a power greater than one. (Recall Chezy-Manning equation). As canal flow rate increases, seepage loss rates will increase as well but become relatively less significant and conveyance efficiency increases.

Observation #3 is a direct consequence of bankflow and storage. During periods of high flow rate, the groundwater table adjacent the canal rises. When the flow rate is substantially decreased, seepage flow from the canal will decrease or even reverse resulting in conveyance efficiencies which approach or even exceed 1.0.

The two days at the end of the study period, during which no farm deliveries were being made, followed immediately after a significant decrease in flow rate caused by operation difficulties with the lateral canal supplying the water to the study canal. The observed conveyance efficiencies for each reach at 20:00 h on September 5, 1982, approximately 36 hours after the sudden decrease are shown in Table 3. Note that the e_c varies with each reach in the range 0.90 to 1.07. The e_c for the entire canal, reach 1 - 6, was 0.90. These values can be expected to decrease with time as the groundwater adjacent the canal gradually drains.

Table 3. Conveyance Efficiencies 20:00 h Sept. 5.

Reach	e_c
1 - 2	0.96
2 - 3	1.06
3 - 4	0.90
4 - 5	1.07
5 - 6	0.92
1 - 6	0.90

Several other important observations were made during the course of the study.

1. The canal operator's estimates of the rate of farm water deliveries were +100% in error. (Heywood (1978) suggested that these errors would exceed 50%). This is not as serious as it may be initially anticipated since the farmer was receiving an adequate water supply and the canal

operator does not employ water measurement when releasing water into the canal. The canal operator will adjust his diversion into the canal until the rate of unused water, called spill, is reduced to a reasonable limit. Considering the fact that the canal operator has almost no way of estimating seepage losses, this is a surprisingly effective method of delivering water.

2. Increases in study canal flow rate, made during the course of a farm delivery being made near WMS #5, resulted in the observed delivery increasing by 25%. This increase was later detected by the canal operator during an inspection of the delivery at which time he returned it to within 5% of its previous value. He did this without aid of any specialized water measurement equipment.
3. An attempt to calibrate an existing check-drop, which employed wood flash boards, located immediately downstream of WMS #3, resulted in observed weir coefficients which varied in an apparently random manner ± 25% from a mean value. It must be concluded that the use of existing check-drops as weirs for estimating flow rates is an inaccurate process but it provides better estimates than the canal operator's judgement.
4. Abrupt changes in flow are rapidly attenuated as the change progresses downstream through the canal. (See Figure 8 (b)). For example, changes in flow made over a period of several minutes 50 meters upstream of WMS #1 required approximately five hours to reach WMS #6 and up to two additional hours for steady canal flow conditions to again be achieved. Knowledge of the times required for these events to occur are critical to the effective operation of the canal.
5. The time required for upstream flow changes to reach downstream

locations varied with each reach studied as shown in Table 4. The average travel time was 0.71 km/hour.

Table 4. Travel Time

Reach	Travel Time (km/hr)
1 - 2	0.54
2 - 3	0.81
3 - 4	0.76
4 - 5	0.76
5 - 6	0.67
1 - 6	0.71

DISCUSSION

Despite the preliminary nature of the study, many important observations concerning the conveyance efficiency and operation of an operating irrigation distributary canal were made. It is recognized that a few of the conclusions reached are circumstantially supported and that some observed phenomenae remain unexplained and subject only to conjecture. Complete interpretation of observed efficiencies, however, require more and better quality information.

Continuous accurate observations of flow rate are required at locations within the canal and at all farm deliveries. The groundwater adjacent to the study canal should be completely described and monitored continuously during the study. In order to explain much of the groundwater dynamics, meteorological data and farm irrigation practices must be recorded as well.

The groundwater regime adjacent an irrigation canal may be extremely variable and dynamic. Bouwer (1969) identified three basic types as shown in Figure 9. Seepage processes associated with each of these types are different. The dynamics of the groundwater table as affected by the raising and lowering of canal water level are shown in Figure 10. The variation in seepage losses from the canal is apparent. Groundwater regimes illustrated in Figure 11 must also be identified. In this instance the canal is operating as an interceptor drain. Precipitation and irrigation adjacent to the canal will also affect seepage loss rates. These effects are illustrated in Figure 12. Under conditions of high water table and the presence of phreatophyte vegetation, that is, plants whose roots can extract water directly from the saturated groundwater zone, canal loss rates can exhibit a seasonal and diurnal variation. This is supported by Meyboon (1967) and by conversation with several canal operators in Southern Alberta. The phenomena is illustrated in Figure 13. During the day phreatophyte transpiration causes the level of the ground water table relative to the canal to decrease. At night the canal recharges the groundwater and the groundwater table rebounds.

The impact of canal operation in determining canal water loss rate is obvious when one considers the operations usually associated with making a farm water delivery. In Figure 14 (a) the farm diversion structure, a turnout, is closed and there are no boards in the depth-control structure, a check drop. When a delivery is to be made, water is backed up over the turnout by adding boards to the check, Figure 14 (b). The increased depth not only facilitates the water delivery but also increases seepage losses.

In any rigorous study of irrigation distributary canal performance, the following improvements in study procedure are suggested:

1. The study should continue over an entire irrigation season. This would allow for a wider range of flow rates and flow conditions to be examined.
2. An ongoing analysis of the data collected should be performed. This would allow timely investigation into apparently anomolous data. Short duration studies usually prohibit action of this sort frequently resulting in the late discovery of poor or unuseable information.
3. Complete groundwater investigation in the vicinity of the canal should be performed prior to and during study period. If necessary, continuous monitoring could be performed to provide data essential for interpretation of conveyance efficiency observations.
4. Meteorological data should be recorded near the study canal. The extent of the observation program would depend on groundwater conditions and agricultural practices adjacent the canal.
5. All water measurement apparatus should be mounted on structures dedicated to this function. The structures should not interfere or influence normal canal operations and should be calibrated on site.
6. All canal operations should be logged.
7. Flow conditions on the lateral supplying water to the canal should be continuously monitored.
8. A record of agricultural activity adjacent the canal, particularly irrigation, should be maintained.
9. A complete cross-sectional and longitudinal description of the canal should be developed.

10. Canal vegetation conditions, terrestial and aquatic, should be documented throughout the growing season.

As clearly demonstrated by this study, canal operation and dynamics are complex. Shortcomings in most studies investigating canal performance are usually a result of over simplification on the part of the investigators.

ACKNOWLEDGEMENTS

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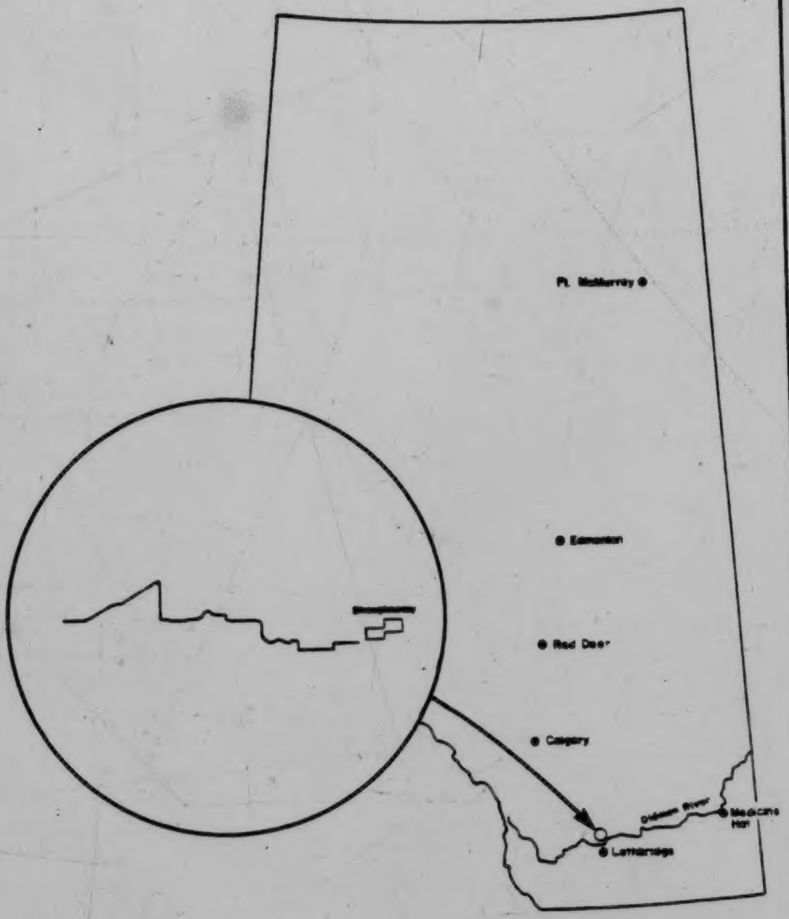

Ft. McMurray
Edmonton
Red Deer
Calgary
Oldman River
Medicine Hat
Lethbridge

FIGURE 1. L.N.I.D. SHAUGHNESSY STUDY CANAL 1982

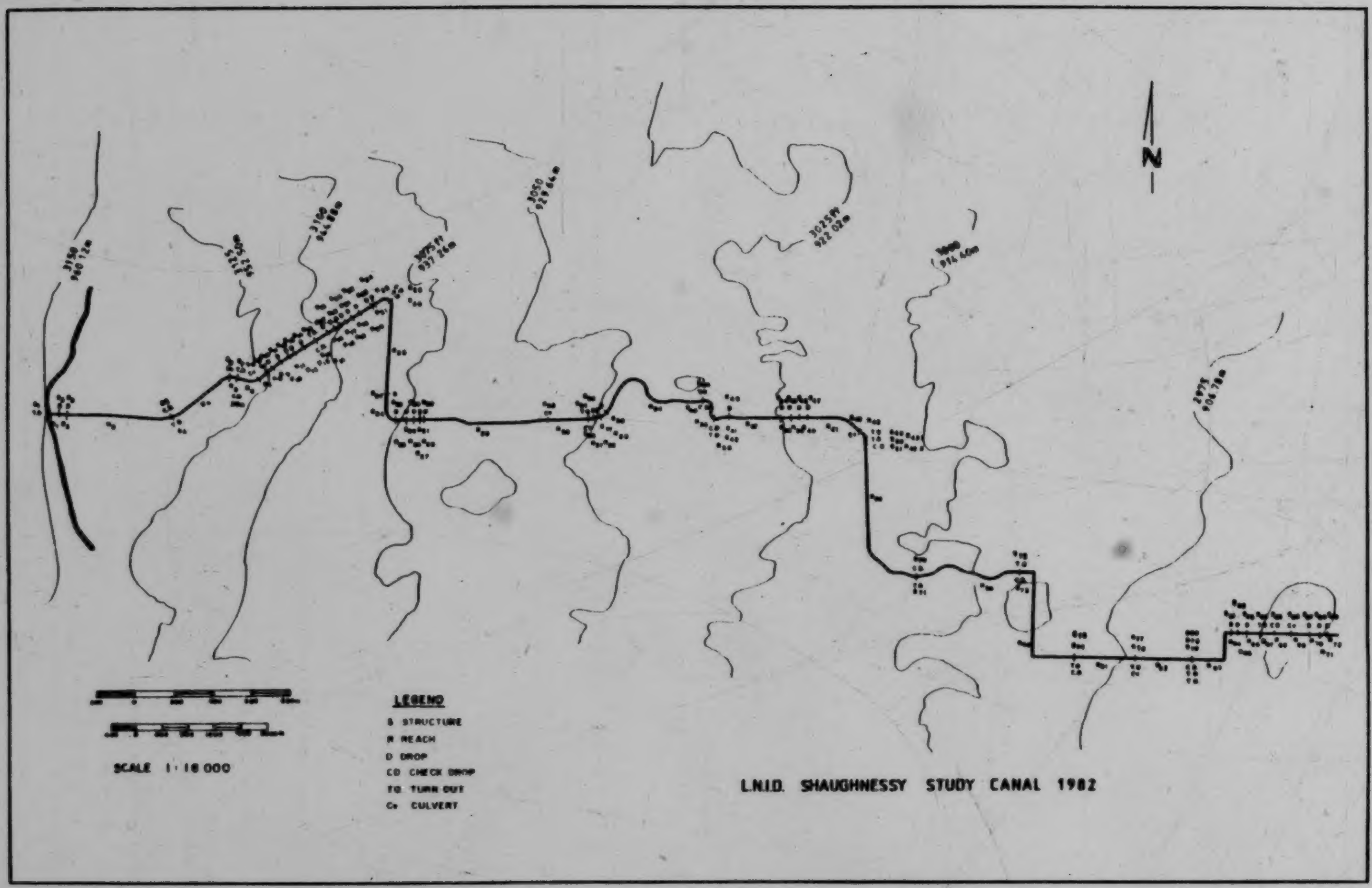
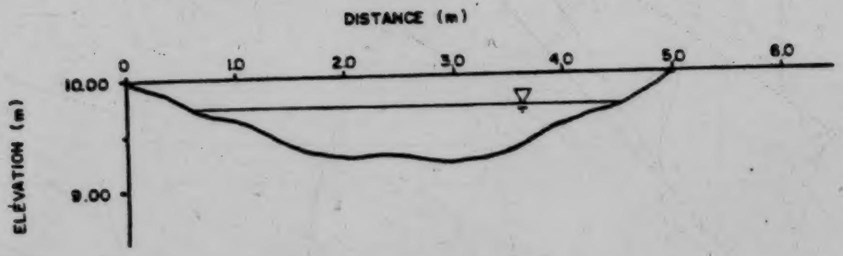
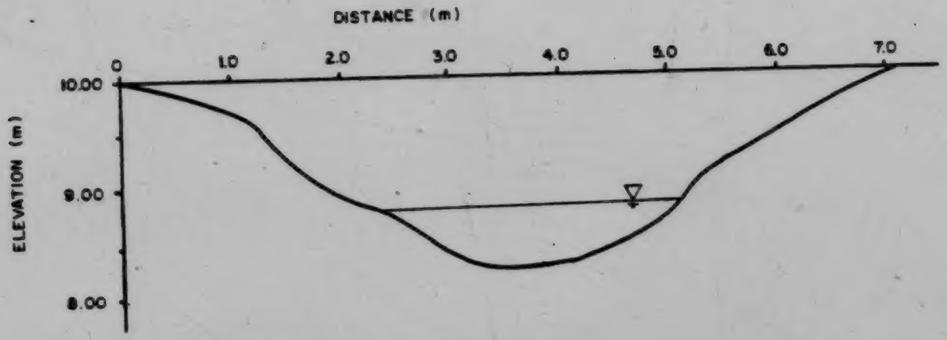

N
3150 960.12m
3125 952.50m
3100 944.88m
3075 ft 937.26m
3050 929.64m
3025 ft 922.02m
3000 914.40m
2975 906.78m
SCALE 1 : 18 000
LEGEND
S STRUCTURE
R REACH
D DROP
CD CHECK DROP
TO TURN OUT
Cv CULVERT
L.N.I.D. SHAUGHNESSY STUDY CANAL 1982

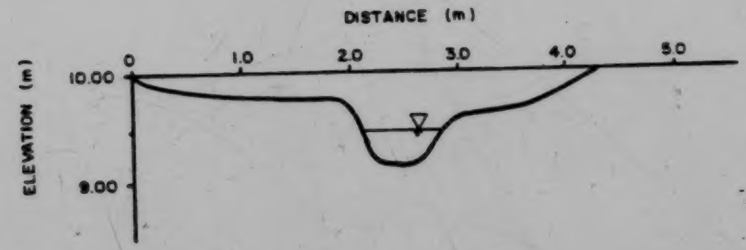
Figure 2. LNID Study Canal


DISTANCE (m)
0
1.0
2.0
3.0
4.0
5.0
6.0
10.00
9.00
ELEVATION (m)
∇

A. CROSS SECTION No. 1

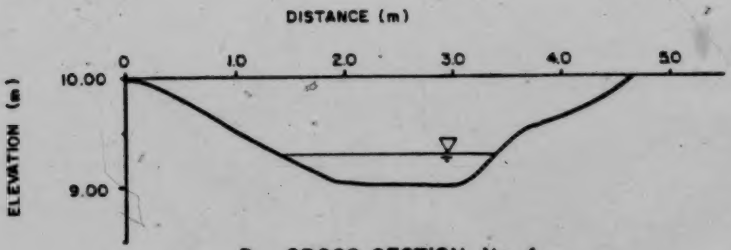

DISTANCE (m)
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
10.00
9.00
8.00
ELEVATION (m)
∇

B. CROSS SECTION No. 2

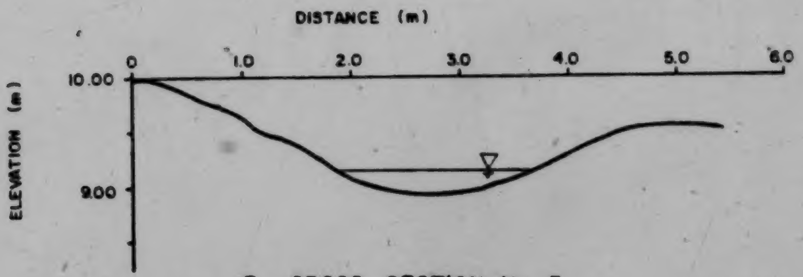

DISTANCE (m)
0
1.0
2.0
3.0
4.0
5.0
10.00
9.00
ELEVATION (m)
∇

C. CROSS SECTION No. 3

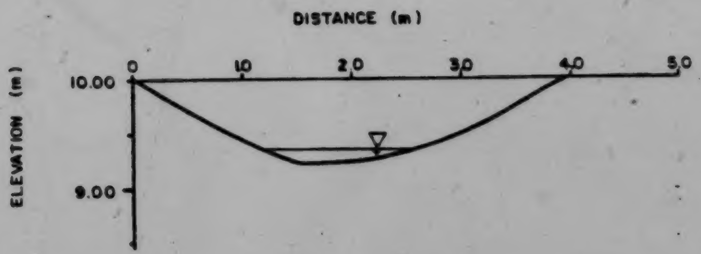
5 FIGURE 3. CANAL CROSS-SECTIONS



D. CROSS SECTION No. 4



E. CROSS SECTION No. 5



F. CROSS SECTION No. 6

FIGURE 3. CANAL CROSS-SECTIONS

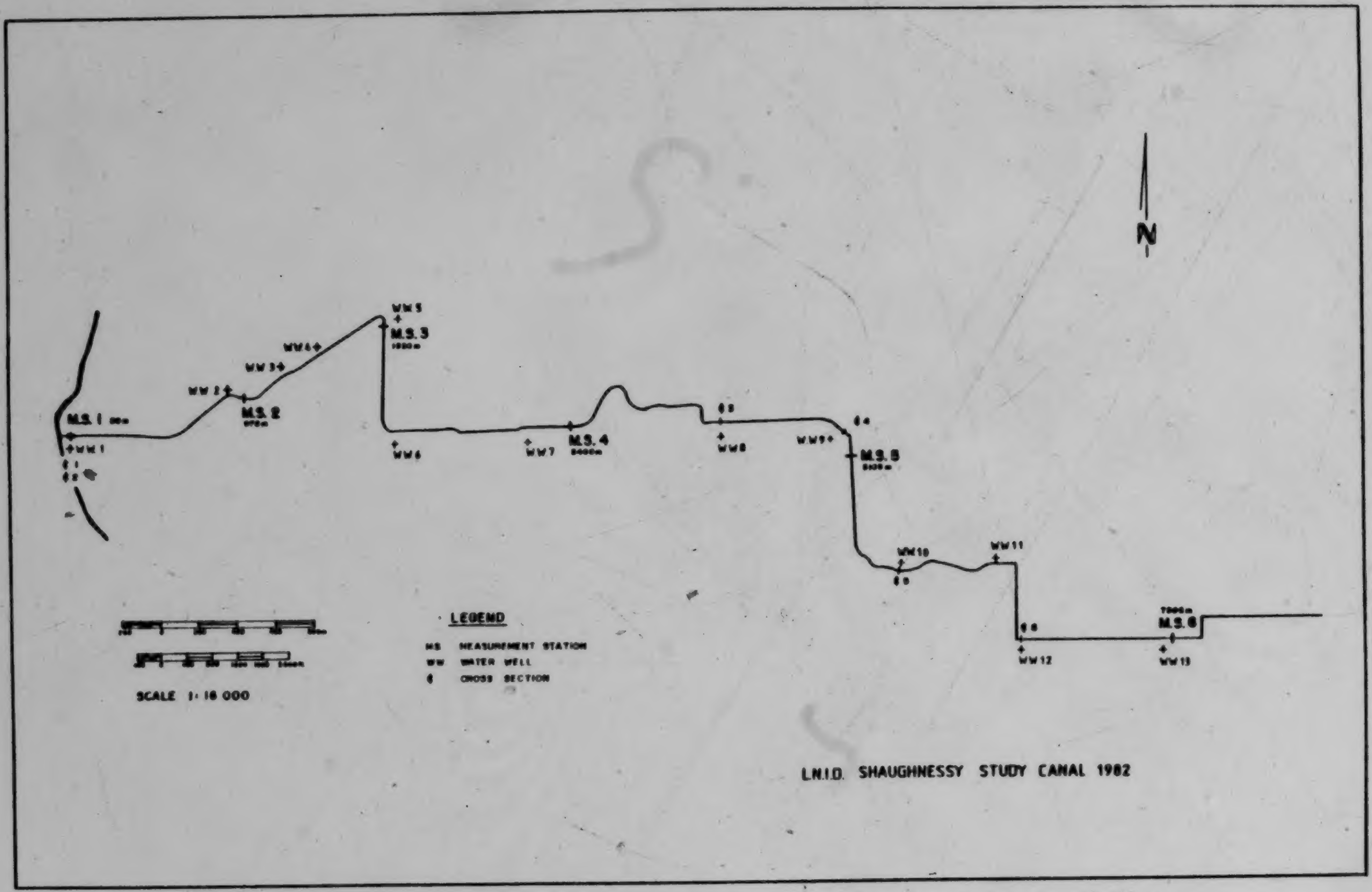

N
M.S. 1
WW 1
WW 2
M.S. 2
WW 3
WW 4
WW 5
M.S. 3
WW 6
WW 7
M.S. 4
WW 8
WW 9
M.S. 5
WW 10
WW 11
WW 12
M.S. 6
WW 13
LEGEND
MS MEASUREMENT STATION
WW WATER WELL
CROSS SECTION
SCALE 1: 16 000
L.N.I.D. SHAUGHNESSY STUDY CANAL 1982

FIGURE 4. LOCATION OF CROSS-SECTIONS, WATER MEASUREMENT STATIONS AND WELLS

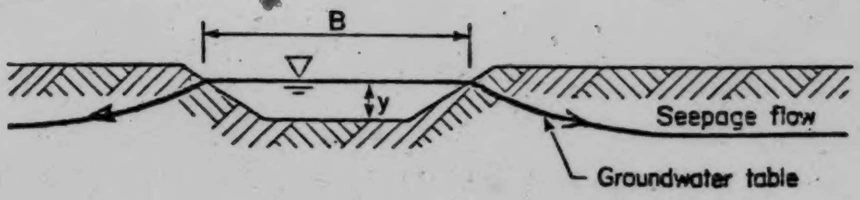

B
y
Seepage flow
Groundwater table

FIGURE 5. RELATIONSHIP BETWEEN CANAL FLOW AND GROUNDWATER ALONG STUDY CANAL

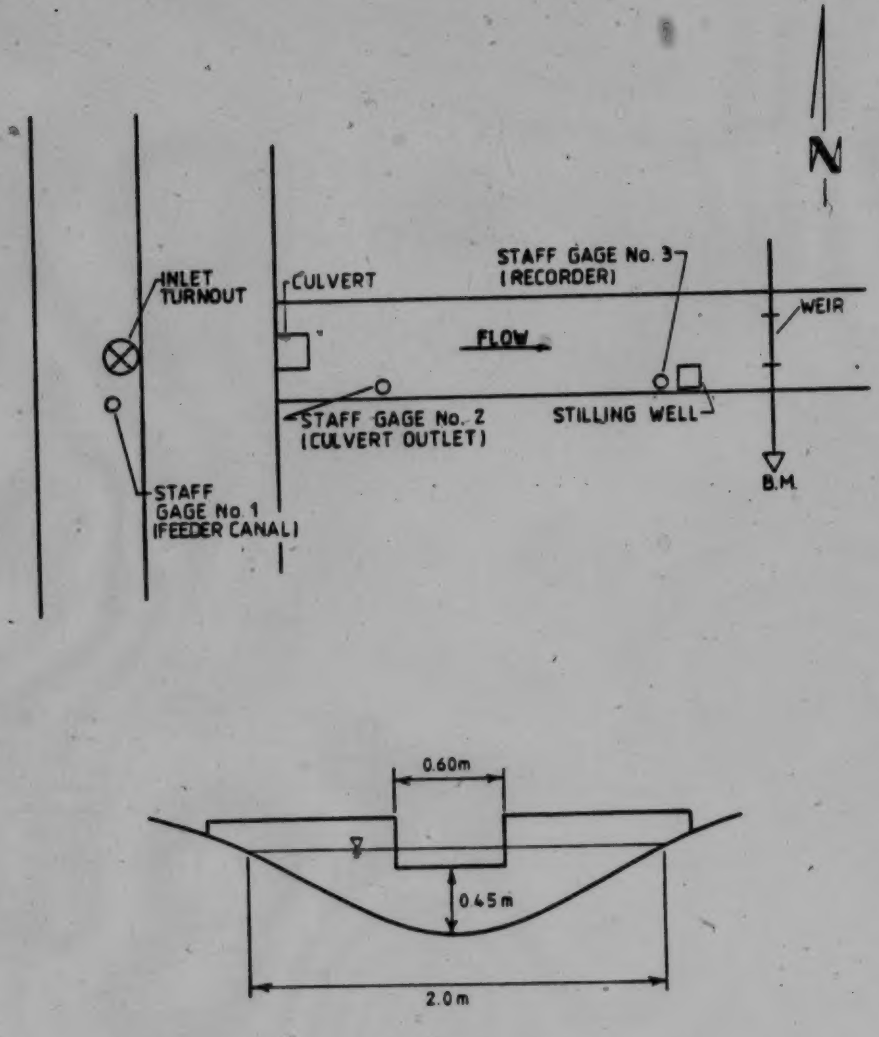

N
INLET TURNOUT
CULVERT
STAFF GAGE No. 3 (RECORDER)
WEIR
FLOW
STAFF GAGE No. 2 (CULVERT OUTLET)
STILLING WELL
B.M.
STAFF GAGE No. 1 (FEEDER CANAL)
0.60m
0.45m
2.0m

FIGURE 6 (A). M.S. 1 SITE DESCRIPTION

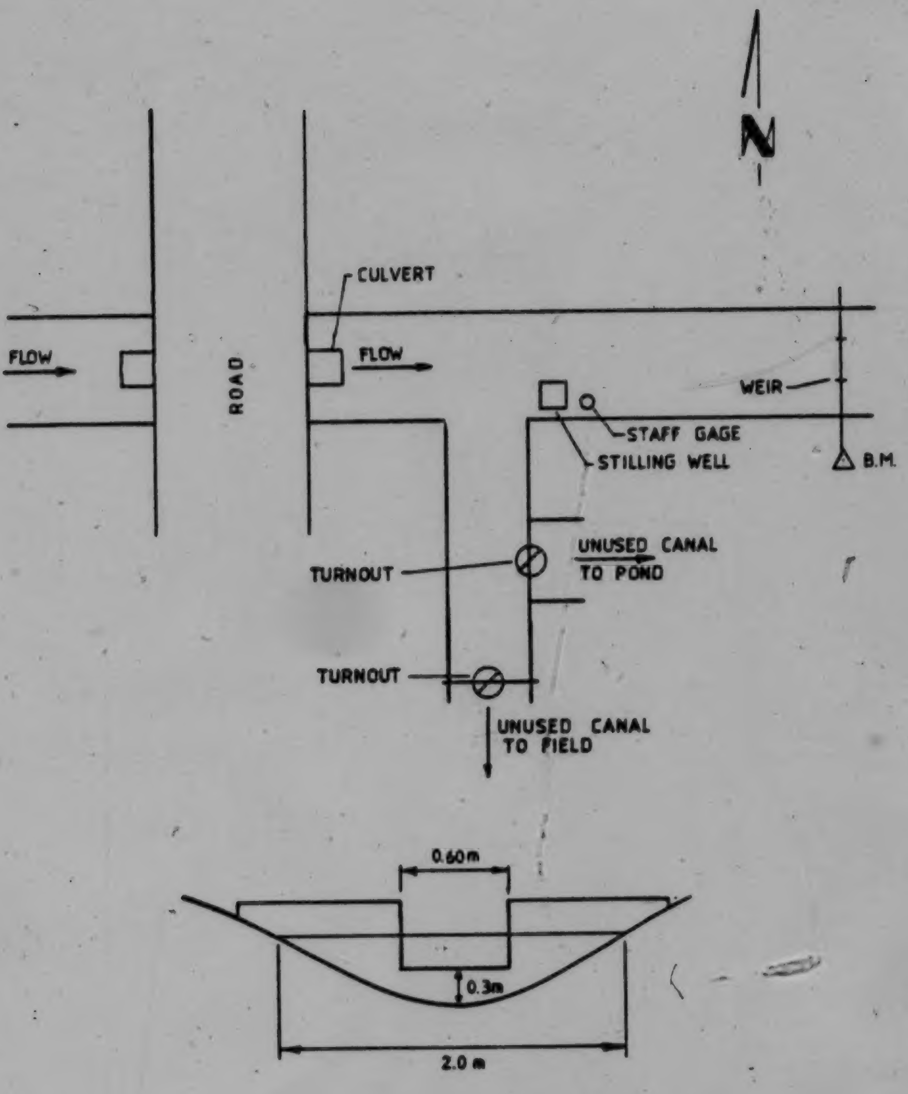

N
CULVERT
FLOW
ROAD
FLOW
WEIR
STAFF GAGE
STILLING WELL
B.M.
UNUSED CANAL TO POND
TURNOUT
TURNOUT
UNUSED CANAL TO FIELD
0.60m
0.3m
2.0 m

FIGURE 6 (B). M.S. 2 SITE DESCRIPTION

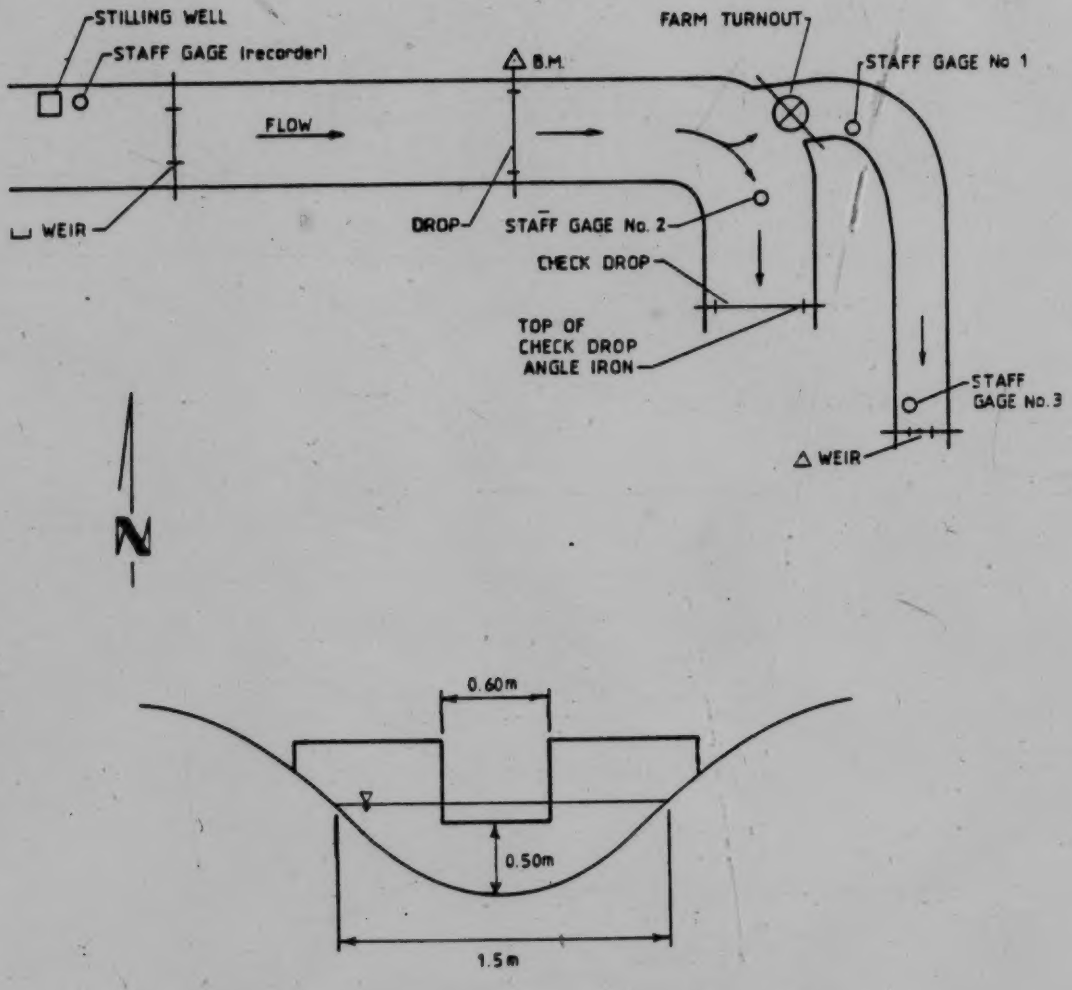

STILLING WELL
STAFF GAGE (recorder)
B.M.
FARM TURNOUT
STAFF GAGE No 1
FLOW
WEIR
DROP
STAFF GAGE No. 2
CHECK DROP
TOP OF CHECK DROP ANGLE IRON
STAFF GAGE No. 3
WEIR
N
0.60m
0.50m
1.5m

Figure 8 (c). M.S. 3 Site Description

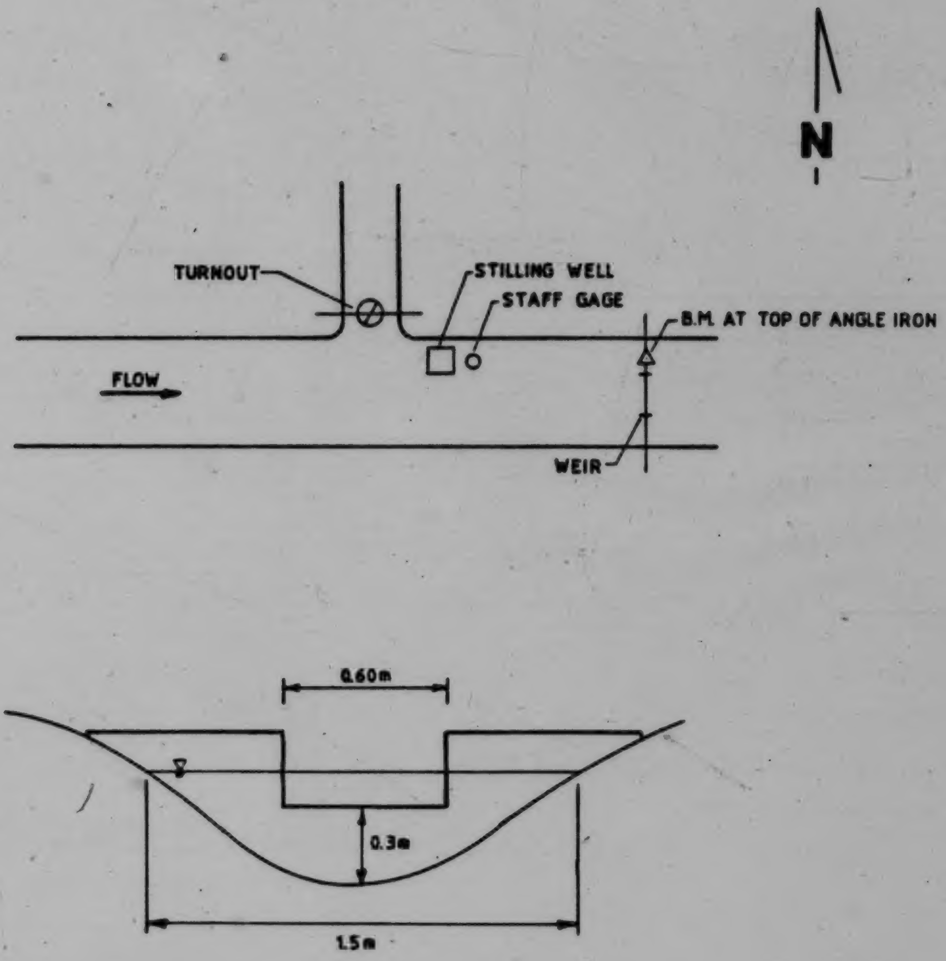

N
TURNOUT
STILLING WELL
STAFF GAGE
B.M. AT TOP OF ANGLE IRON
FLOW
WEIR
0.60m
0.3m
1.5m

FIGURE 6 (D). M.S. 4 SITE DESCRIPTION

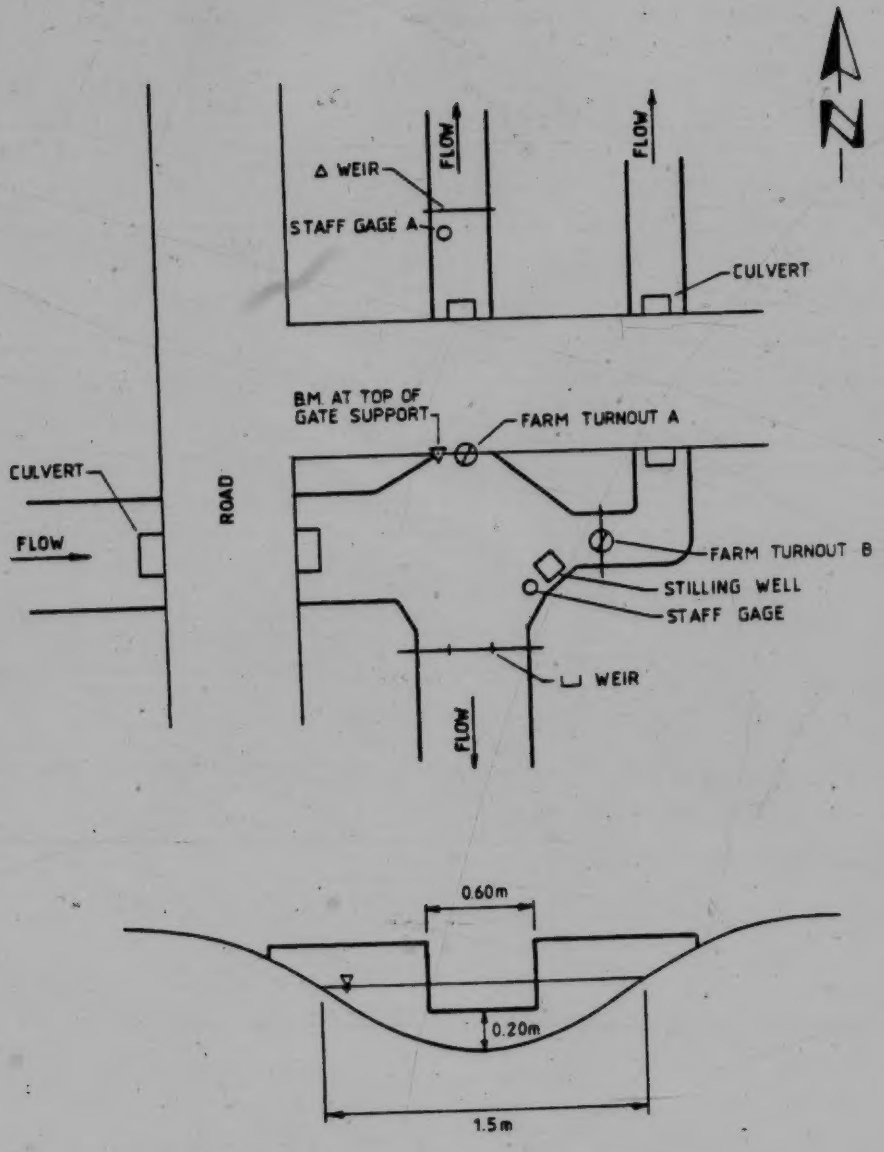

FLOW
FLOW
N
Δ WEIR
STAFF GAGE A
CULVERT
B.M. AT TOP OF GATE SUPPORT
FARM TURNOUT A
CULVERT
FLOW
ROAD
FARM TURNOUT B
STILLING WELL
STAFF GAGE
⊔ WEIR
FLOW
0.60m
0.20m
1.5m

Figure 6 (e). M.S. 5 Site Description

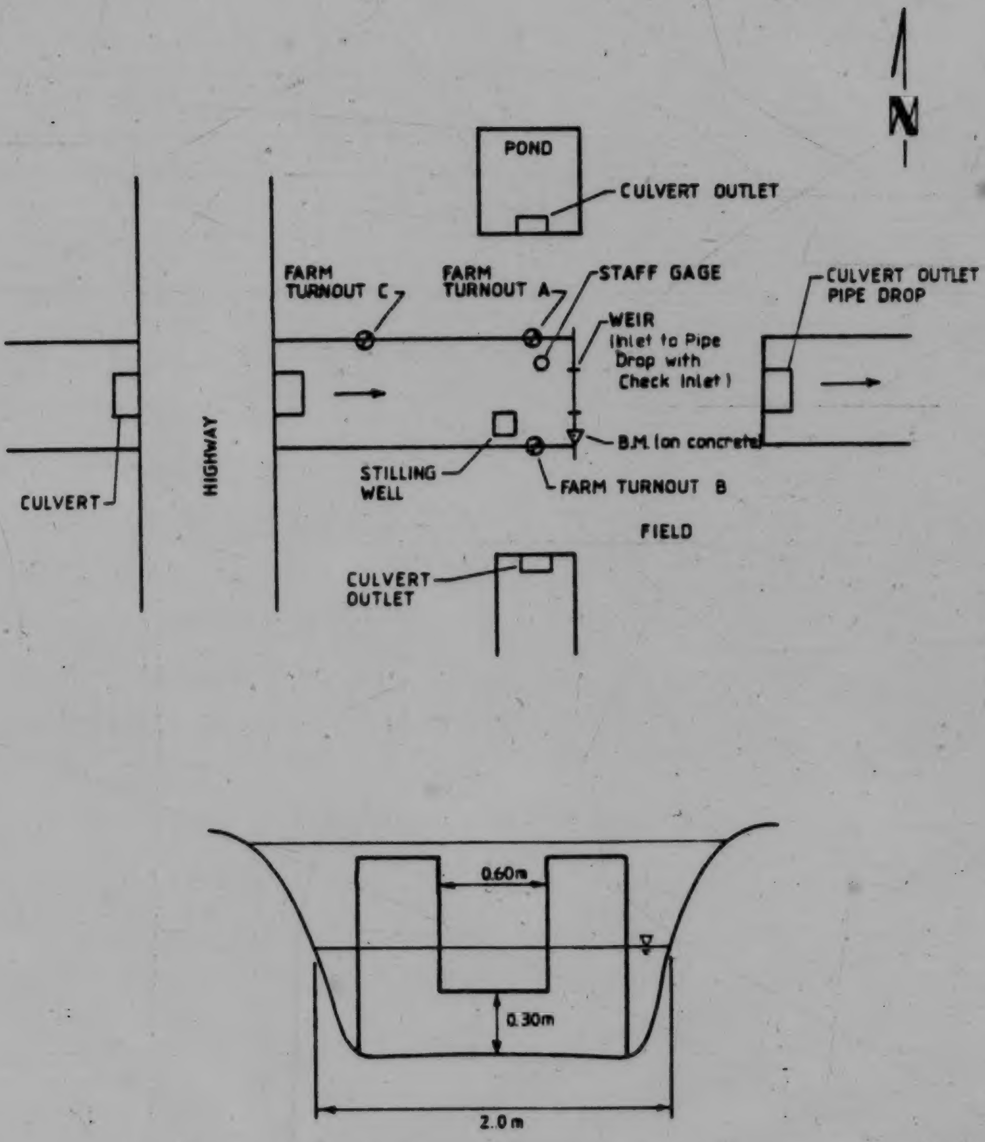

N
POND
CULVERT OUTLET
FARM TURNOUT C
FARM TURNOUT A
STAFF GAGE
WEIR (Inlet to Pipe Drop with Check Inlet)
CULVERT OUTLET PIPE DROP
HIGHWAY
B.M. (on concrete)
STILLING WELL
FARM TURNOUT B
CULVERT
FIELD
CULVERT OUTLET
0.60m
0.30m
2.0m

Figure 6. M.S. 6 Site Description

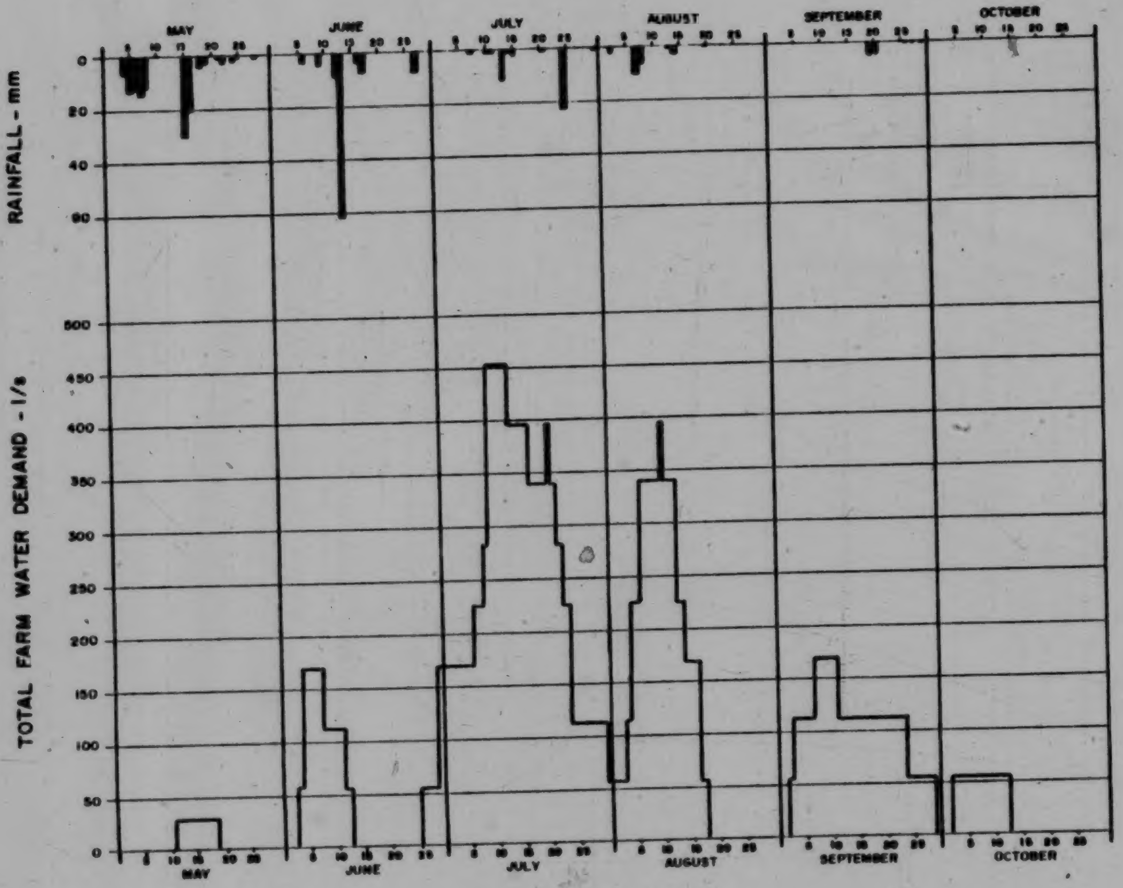
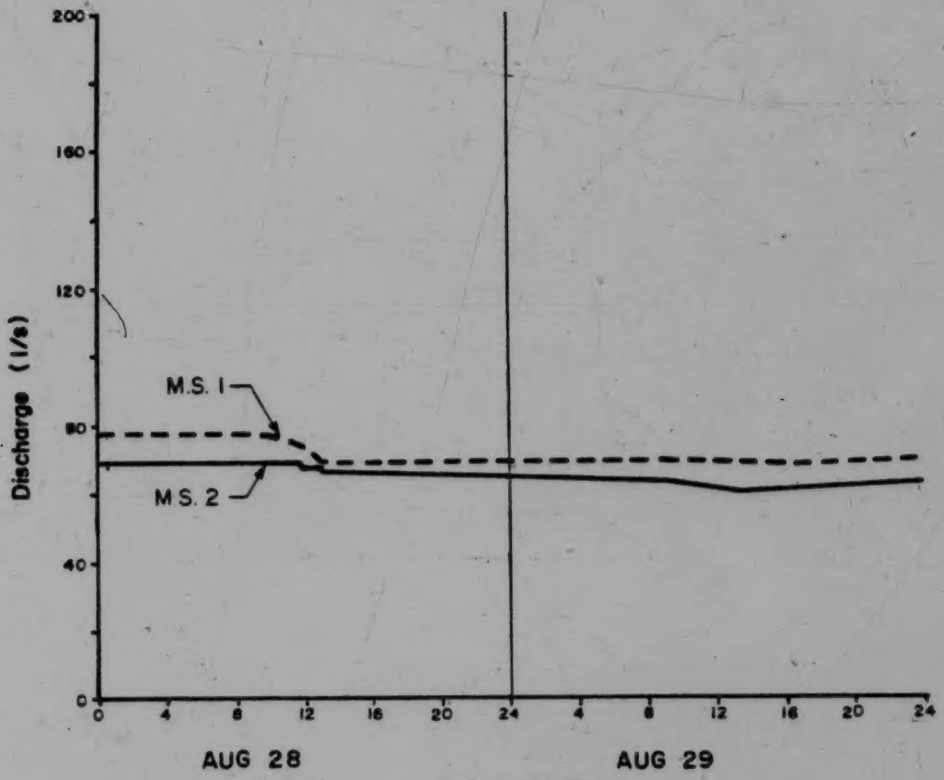
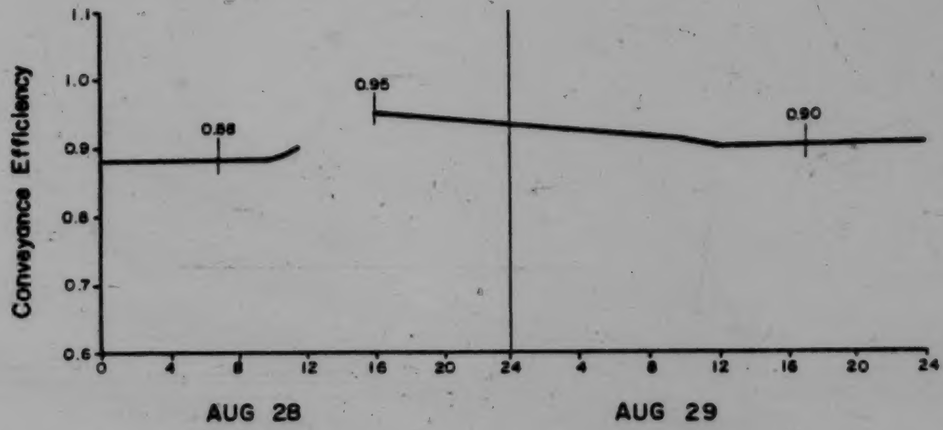


FIGURE 7. FARM WATER DEMAND AND RAINFALL - 1981

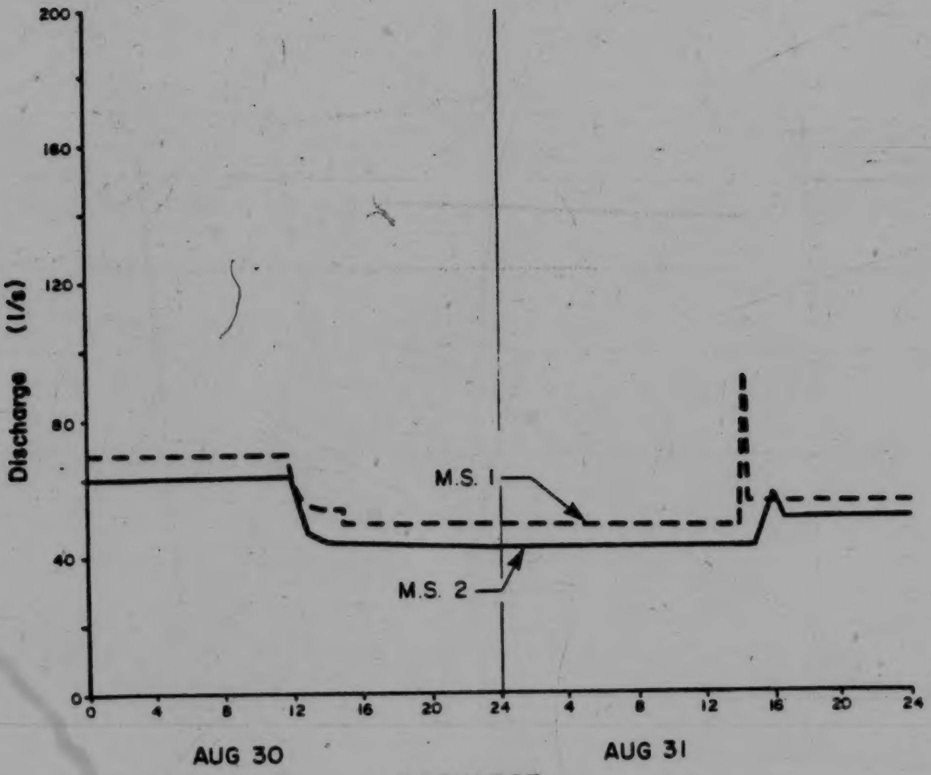


DISCHARGE



IRRIGATION CONVEYANCE EFFICIENCY

FIGURE 8 (A). DISCHARGE WMS #1 AND WMS #2 AND ASSOCIATED E_C



DISCHARGE

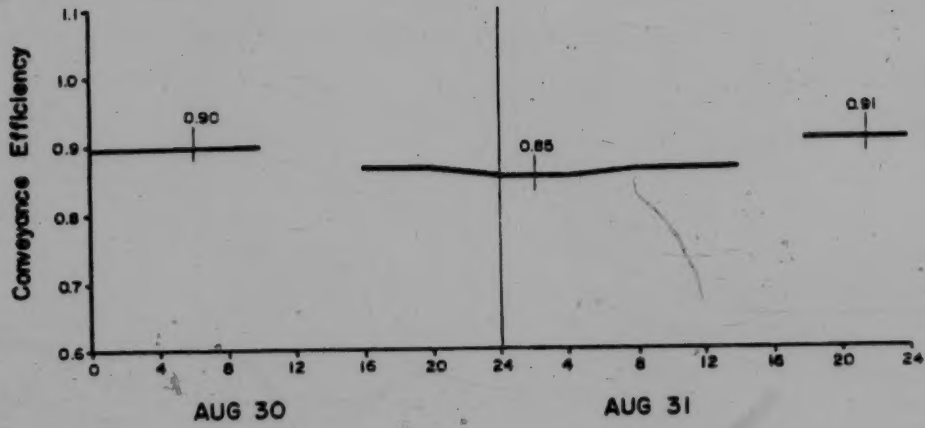
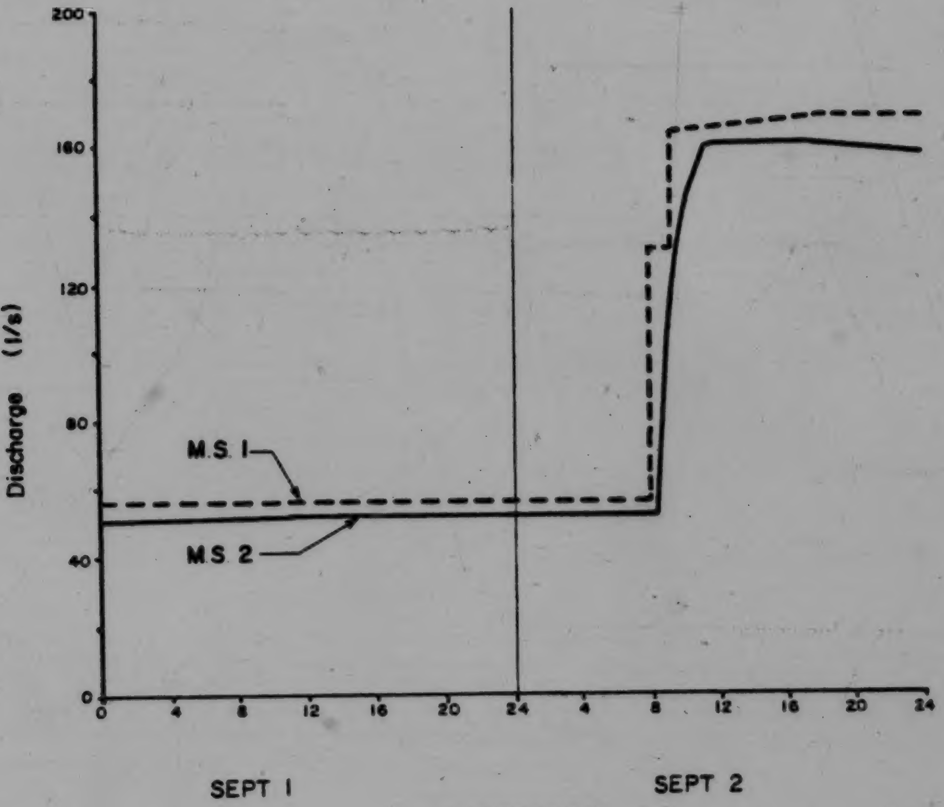


FIGURE 8(B). IRRIGATION CONVEYANCE EFFICIENCY



DISCHARGE

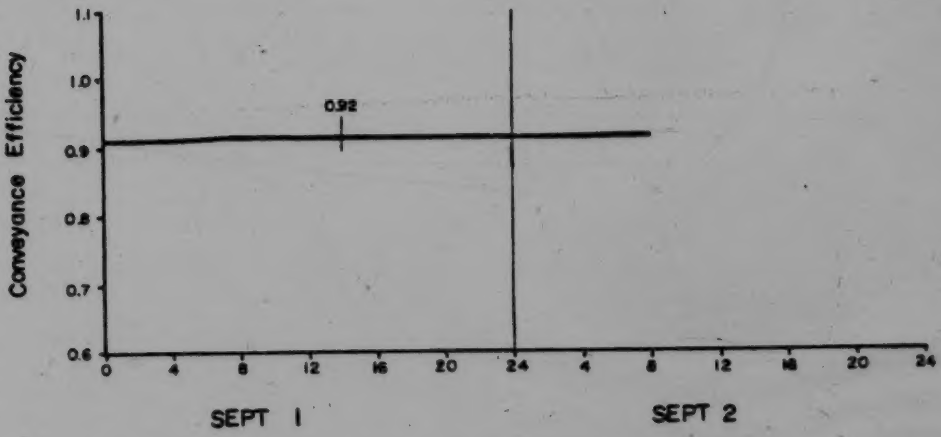
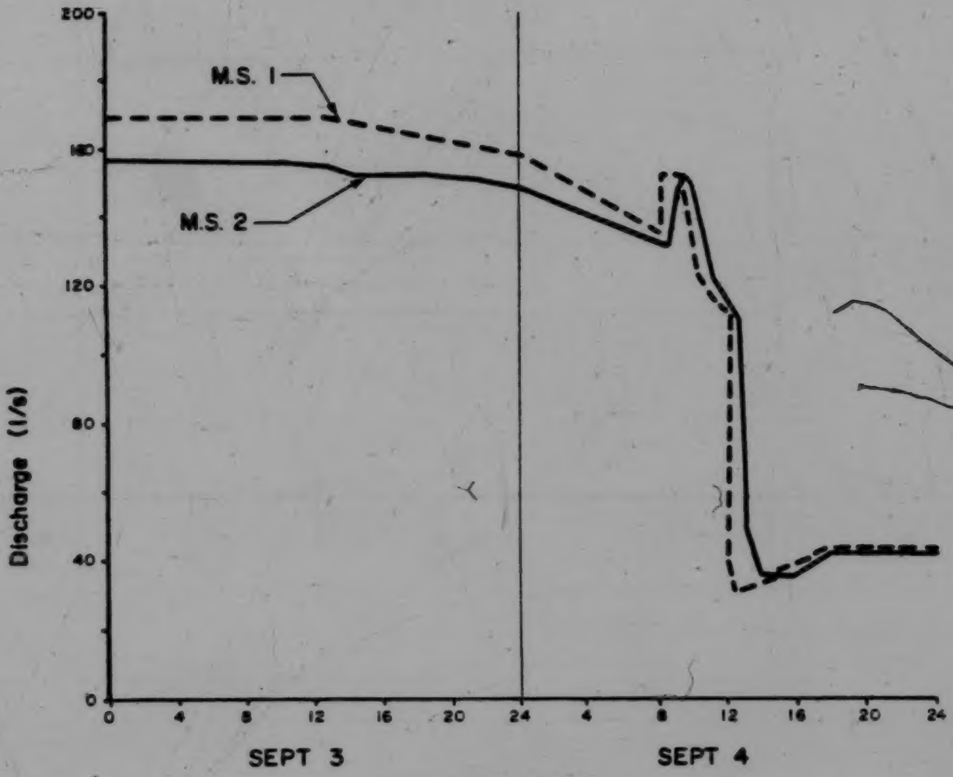


FIGURE 8(c). IRRIGATION CONVEYANCE EFFICIENCY



DISCHARGE

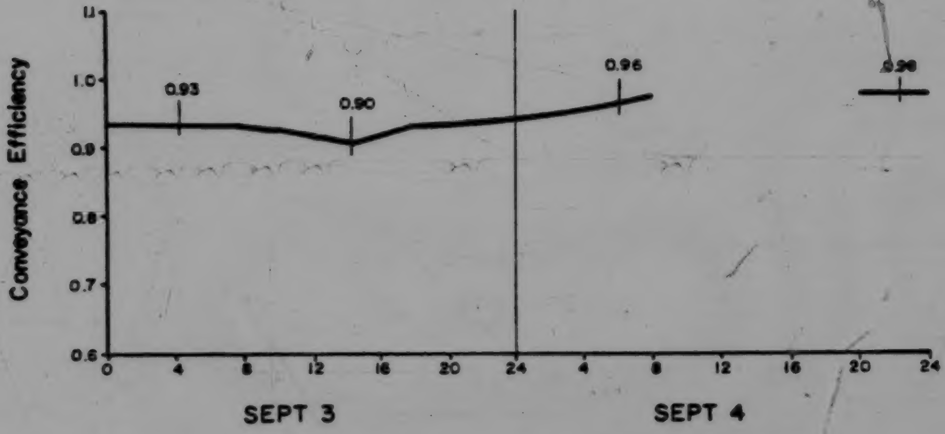
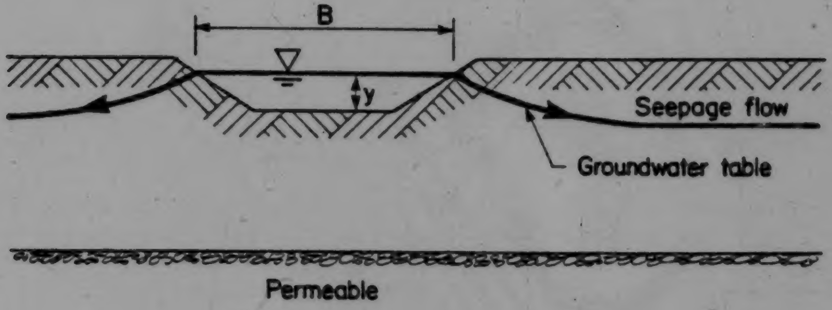
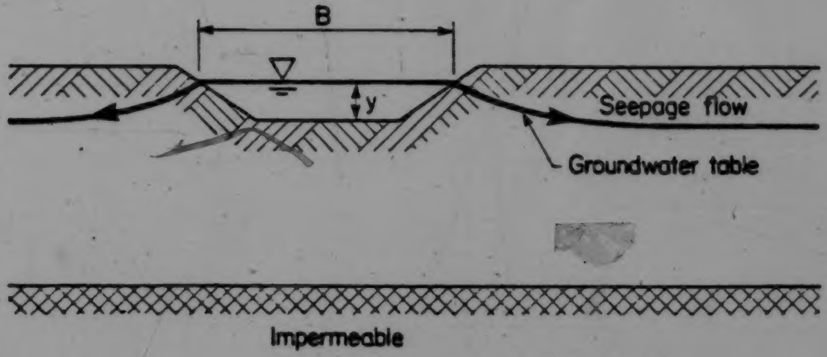


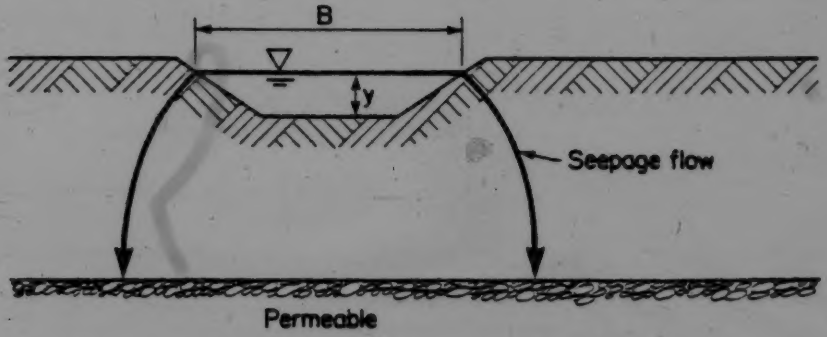
FIGURE 8(D). IRRIGATION CONVEYANCE EFFICIENCY


B
y
Seepage flow
Groundwater table
Permeable

(A) TYPE A

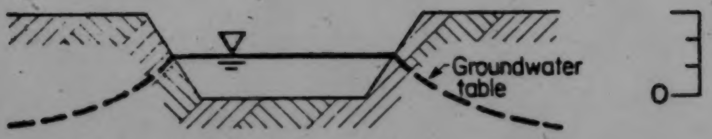

B
y
Seepage flow
Groundwater table
Impermeable

(B) TYPE B

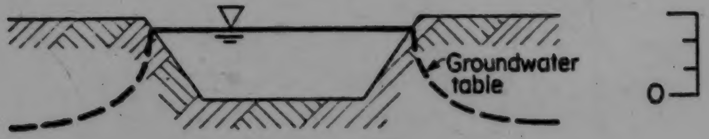

B
y
Seepage flow
Permeable

(C) TYPE C

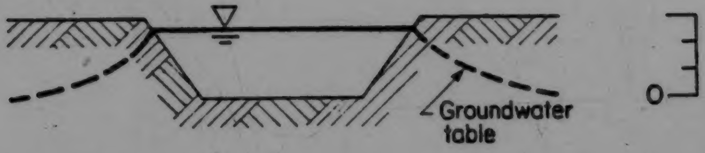
FIGURE 9. RELATIONSHIPS BETWEEN GROUNDWATER AND CANAL FLOW CONDITIONS


Groundwater table
0

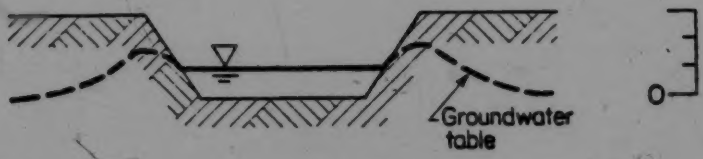
(A) STEADY-STATE


Groundwater table
0

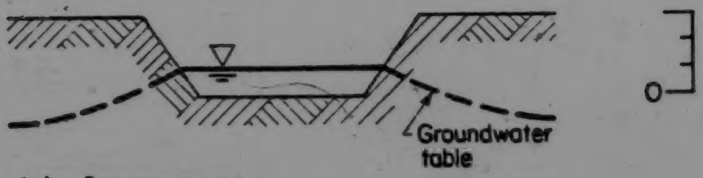
(B) ABRUPT INCREASE IN FLOW


Groundwater table
0

(C) STEADY-STATE


Groundwater table
0

(D) ABRUPT DECREASE IN FLOW


Groundwater table
0

(E) STEADY-STATE

FIGURE 10. RELATIONSHIP BETWEEN GROUNDWATER AND STEADY AND UNSTEADY CANAL FLOW CONDITION

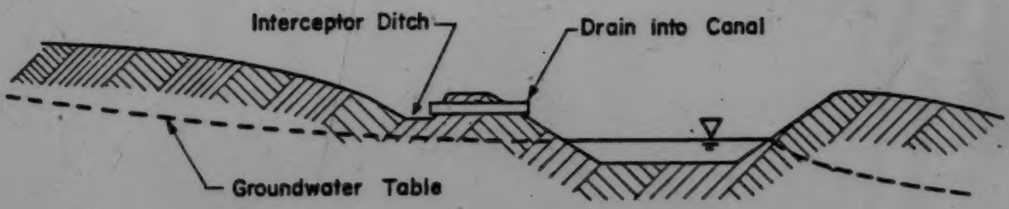
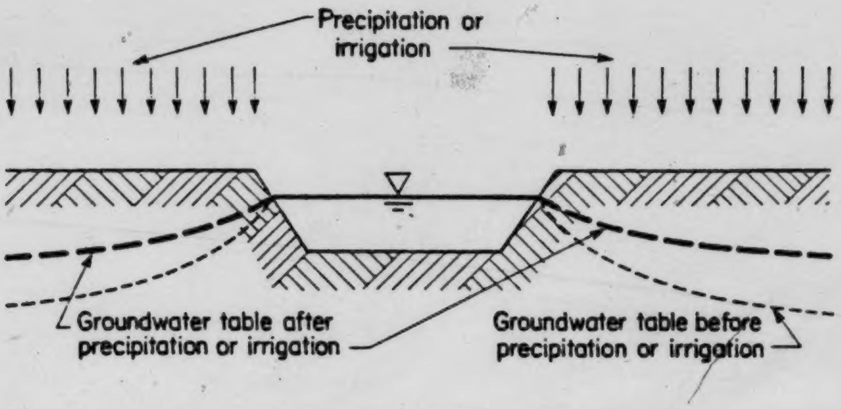
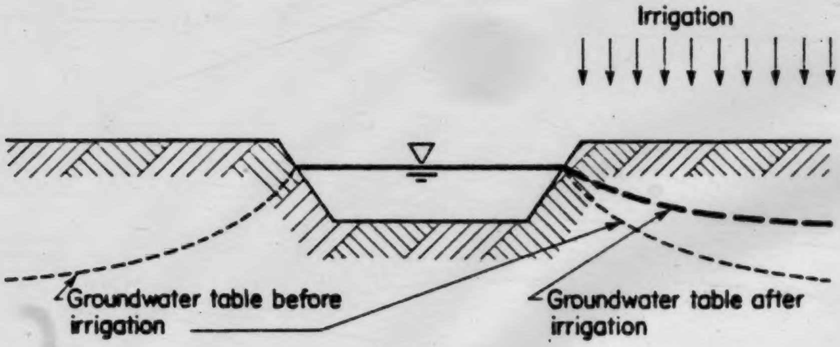

Interceptor Ditch
Drain into Canal
Groundwater Table

FIGURE 11. CANAL ACTING AS AN INTERCEPTOR DRAIN.


Precipitation or irrigation
Groundwater table after precipitation or irrigation
Groundwater table before precipitation or irrigation

(A) IRRIGATION AND PRECIPITATION BOTH SIDES OF CANAL


Irrigation
Groundwater table before irrigation
Groundwater table after irrigation

(B) IRRIGATION ON ONLY ONE SIDE OF CANAL

FIGURE 12. EFFECT OF PRECIPITATION AND IRRIGATION ON GROUNDWATER

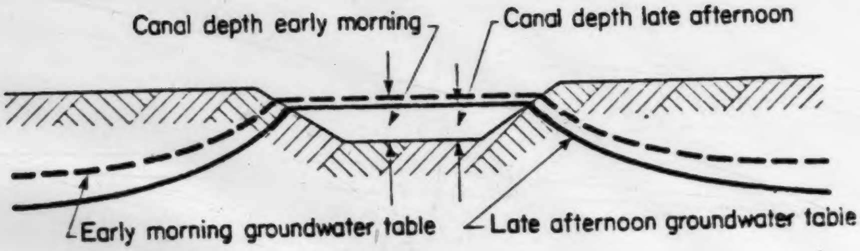
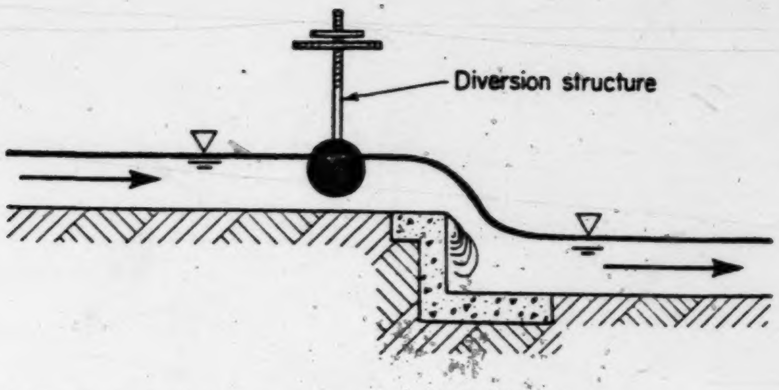
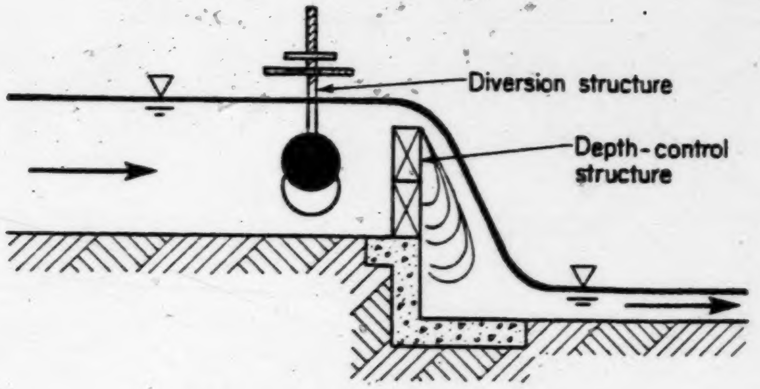

Canal depth early morning
Canal depth late afternoon
Early morning groundwater table
Late afternoon groundwater table

FIGURE 13. EFFECT OF PHREATOPHYTE VEGETATION TRANSPIRATION ON GROUNDWATER AND CANAL FLOW CONDITIONS.


Diversion structure

(A) Farm turnout and check before delivery


Diversion structure
Depth-control structure

(B) Farm turnout and check during delivery

Figure 14. Effect of the operation of farm turnout and check structure on water depth on canal